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Conceptual Design to Recycle Water in Post Panamax Locks

CIVIL ENGINEERING WORKS
Cost Estimation
Final Report

R-CW-02

Tractebel Development Engineering
TECHNUM
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Compagnie Nationale du Rhône

CENTRO DE RECURSOS TECNICOS
AUTORIDAD DEL CANAL DE PANAMA

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1 Executive Summary

This report deals with the second phase of the conceptual design to recycle water in Post Panamax locks, at the Pacific side of the Panama Canal. This second phase concerns the electro-mechanical and civil engineering work, and cost estimation. The first phase deals with the hydraulic components of such system and are reported in R-HY-002-B.
The following three scenarios have been investigated:

- Direct Pumping (from Pacific Ocean to Gatun Lake)
- Semi Direct Pumping (from a lower pond to Gatun Lake)
- Pond to Pond Pumping (from a lower to an upper pond.

These 3 scenarios have been worked out for 4 lock configurations and three operational modes. The three scenarios are the following:

- Triple lift lock system with 3x3 Water Saving Basins (WSB)
- Triple lift lock system without WSB
- Double lift lock system with 2x2 WSB
- Double lift lock system without WSB

Following modes of operation have been taken into account:

- 1-5 lock operations a day
- 5-10 lock operations a day
- 10-15 lock operations a day

It has been shown during the first phase of the study that all scenarios are feasible from the technical point of view. Other constraints, such as environmental, are not considered nor evaluated as they are not part of the scope of work.

The electromechanical components required for the pumping system are highlighted in reports R-EM-01 and 02. These components consist mainly of vertical tube pumps , butterfly valves, pumping stations with electrical equipment and monitoring equipment. An estimation is given of the energy consumption in function of the required pumping rates.

This report R-CW-02 on civil works and cost estimation describes the different civil engineering components of the systems, which are mainly the following:

- Upper pond with dam and spillway
- Lower pond
- Pumping stations

- Penstocks
- Connecting culverts between ponds and locks (2)

Finally, a construction schedule and cost estimation has been prepared.
The base line scenario with direct pumping is definitely the less complicated and can be realized in a construction period of nearly two years. It does not require adaptations to the lock system as it pumps independently water from the Pacific to Gatun Lake.
The two other base line scenarios are more complicated as the pond(s) have to be connected to the lock chamber(s), which requires an adaptation to the lock culvert system, and this has to be integrated in the lock construction. This means that any decision on this type of recycling has to be taken before basic and detailed design.
Both scenarios can be realized in a period of nearly 3 years, but demand important activity in the close vicinity of the locks, which will be in operation. Boring techniques have therefore been envisaged to realize the connecting culverts.

As far as the penstocks are concerned, as they are an important part of the investment due to the length, have been foreseen as steel large diameter pipes as this is the most common practice. Nevertheless, since steel prices depend on economical factors that are difficult to predict and seem to become unfavorable at the moment of editing of this report, it should be noted that the penstock can as well be constructed in reinforced concrete.

The cost estimation for the total of construction work and electromechanical equipment is given below for the three base line scenarios and the two lock considered configurations, with and without WSB, and assuming maximum operation mode:

	Lock Configuration			
Base line scenario	3Lift + WSB	3Lift without WSB	2Lift with WSB	2Lift without WSB
Direct Pumping	$34.328.362	$78.414.978	$55.055.223	$108.923.550
Semi Direct Pumping	$176.973.087	$214.902.238	$206.345.305	$243.426.601
Pond to Pond Pumping	$274.658.459	$315.482.051	$317.186.663	$361.891.572

Maintenance end operation cost (M&O) can be estimated yearly at 1% of the investment cost.

2 Upper Pond with Dam and Spillway

2.1 GENERAL CONSIDERATIONS

In the Phase I study report (R-HY-002), for the baseline scenario 3 with pond to pond pumping, a very convenient location for the upper pond was identified on the Rio Cocoli, to the west of the new Post Panamax locks.

Picture taken at future dam location

Indeed, the pond will thus be located at the same side of the locks as the lower pond (see further), which avoids a difficult crossing of the penstock beneath the locks or the canal. Furthermore, no additional space will be required upstream in the locks area to construct such a reservoir.

Construction of a simple RCC dam on the river Cocoli with spillway will be sufficient to create a natural reservoir along the former riverbed which reaches further inland to the west.

The dam location is indicated on drawing CW-012, and has been determined according to the topographic data furnished by ACP. However the topographic data do not cover the entire reservoir area, and additional topographic survey will have to be performed for detailed design purposes. At this moment, assumptions were made based on older maps, although sufficiently accurate to estimate areas and volumes. The value of these figures varies with the future lake level, and are indicated on the graphs of appendix 2 of this report. In the hydraulics report R-HY-002 has been explained that the level of the lake will vary between a minimum of +27.00m PLD and a maximum of +29.2m PLD during operation conditions. The level may be lower for maintenance reasons, or higher during flooding (+30.68m PLD at PMF conditions).

View along Rio Cocoli – upstream of future dam location.

2.2 HYDRAULIC DESIGN

2.2.1 HYDROLOGY AND RESERVOIR TYPICAL CURVES

The Rio Cocoli synthetic flood hydrograph has been prepared based on the topographical characteristics of the dam catchments area and on four typical precipitation events characterized by their intensity and duration in relation with the following flood return periods:

- 20 years
- 50 years
- 100 years (Q100)
- Probable Maximal Flood (PMF)

The adopted method gives for each flood return period a triangular shaped hydrograph that is subsequently smoothened using the Soil Conservation Survey (SCS) dimensionless hydrograph method.
The Appendix 1 shows the 4 calculation sheets of the synthetic hydrographs for the different flood return periods.
The Rio Cocoli Reservoir characteristics are given in Appendix 2, which shows the catchments area controlled by the Rio Cocoli dam, together with the reservoir surface and capacity curves versus the reservoir level.

2.2.2 SPILLWAY TYPE

The simple free overflow spillway generally gives the most economical spillway type in combination with an RCC dam.
It is this spillway type that has been selected for the Rio Cocoli Dam, adopting a straight sill above a portion of the dam crest.
With this type of spillway the discharge rate is a function of both the head above the spillway crest level and its width. The width of the spillway is therefore adjusted considering that:

- The shorter the width of the spillway is, the higher the head will be during evacuation of the flood discharge, and accordingly the remaining dam section will have to be higher as well, but,

- The higher the head above the crest is, the bigger will be the temporary retention volume in the reservoir above the crest level with the corresponding attenuation effect of the peak flood discharge through the reservoir.

This attenuation effect is evaluated in the so called Flood Routing calculation giving the spillway released hydrograph as a function of the reservoir entering flood hydrograph, the spillway hydraulic characteristics and the reservoir shape.

2.2.3 SPILLWAY HYDRAULICS

The spillway is designed to control and evacuate the floods under specific conditions.
In the present stage of studies, two flood events have been considered:

- the design flood for which the spillway is specifically designed and dimensioned and which corresponds to the 100 years return period flood
- the PMF (probable maximum flood) for which the spillway is verified

The Flood Routing calculations commented in the previous paragraph have been performed for these two floods.

The hydraulic law adopted for the spillway is the following:

$$Q = C\,L\,(H)^{3/2}$$

With Q= Discharge in m^3/s
L = Spillway width in m
H= Water head above the crest
C= spillway weir dimensionless hydraulic coefficient = 2.033

A satisfactory compromise in terms of flood peak attenuation and required non overflow dam height as discussed in the previous paragraph is given for a spillway crest fixed at Elevation +29.50m PLD with a width of L=100m considering as well the topographical shape of the valley and the space needed for the right bank intakes in the dam body.

The Appendix 3 gives the results of the Flood Routing for the 100 years return period whereas the Appendix 4 gives the results of the Flood Routing for the PMF .

It is assumed that the flood routing starts with a full reservoir at El. +29.50m PLD.

The main results are summarized in the table here below:

	Q_{100}	Q_{PMF}
Spillway crest level	+29.50m PLD	+29.50m PLD
Dam crest (non overflow section)	+31.50m PLD	+31.50m PLD

Reservoir entering peak discharge	262 m^3/s	342 m^3/s
Spillway outgoing peak discharge	204 m^3/s	274 m^3/s
Maximum Head	0.98m	1.18m
Maximum Reservoir Water Level	+30.48m PLD	+30.68m PLD
Freeboard in non overflow section	~ 1m	~ 0.80 m

Considering the limited size of the reservoir, the maximum fetch distance and the fact that the dam is a concrete dam (not subjected to erosion when flooded), the freeboard left for both extreme events is considered adequate.

2.3 GEO-TECHNICAL DESIGN

2.3.1 GEOLOGY

The geological site conditions have been assessed by an experienced ACP geologist who is familiar with the situation in the area, especially in the vicinity of the existing and future locks. Although no site investigation (borings, penetration tests) has been undertaken thus far, following geological interpretation was prepared by ACP :

A visit to the site was conducted in May 2003, to verify the existing type of rock and to estimate the thickness of the meteorized soil and rock cover. Although we were not able to locate rock exposure, we could observe the existence of a great deal of angular and sub-angular round edges with an average diameter of 20 cm composed of basalt, in the foundation on the right side between the road and the river. Because the site under observation looks very disturbed after the land removal work carried out for the construction of the road, it is likely that this material was removed from top to bottom.

From the road up, the material is found in its place, although it is in a very advanced state of meteorization, which practically leaves it as soil. Rock exposure was not found in this area either. The fact that it seems like there is no rock exposure either in rivers or in road slopes, along with the advanced level of meteorization found in cuts on the road, suggests that the rock in good shape could be found at an average 10 m deep.

This estimate is in agreement with findings of samples drilled for the design of the Cocoli River bridge, where the average cover is found around 10 m if we do not take into consideration the filling material placed in certain areas during the excavation in 1943.

The presence of basalt is also in accordance with the description of this area in the Panama Canal map on a scale of 1:100,000, where intrusive and extrusive basalt from the middle and upper Miocene are found. However, in lower elevations, more or less at the level of the river channel, agglomerates are found from the Pedro Miguel Formation, which belong to the lower Miocene, and then the basalt is found on this formation. This basalt is intercepted to the North East by the Cocoli River fault and the Pedro Miguel fault.

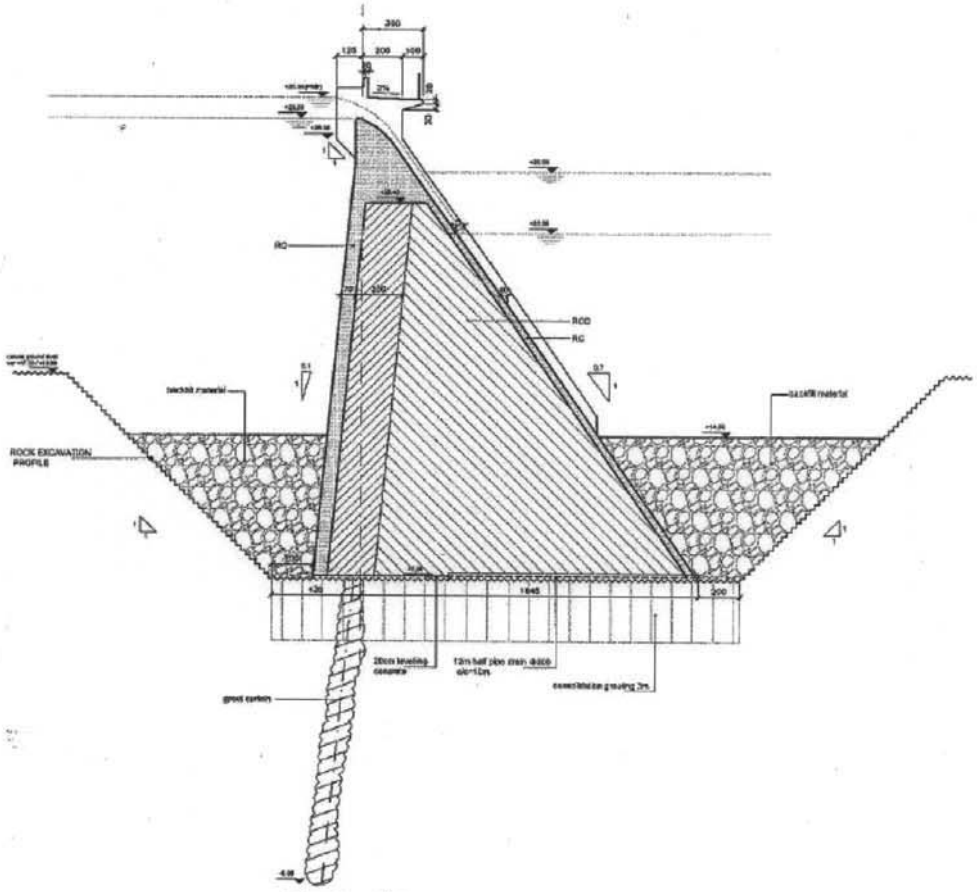
Of the laboratory analyses conducted on the basalt and the agglomerate for the conceptual design of the third set of locks, we may conclude that resistance to the simple compression of these two rocks is somewhat similar and that they could be around the 40 Mpa. In general, after observation of the drilling in surrounding areas, the basalt tends to present itself fractured, whereas the agglomerate is massive. Permeability in both rocks tends to be low due to the fact that basalt fractures are often filled with clayey minerals and agglomerate tends to have few fractures.

Although it is clear that sound rock may be found at 10m below natural ground level, it is obvious that extensive soil investigation testing will have to be performed prior to the detailed design of the dam structure. For conceptual design purposes however, it will be assumed that the foundation level of the dam will be situated at some 10m below actual ground surface.

2.3.2 GENERAL STABILITY

A. Dam layout

The dam structure is shown on drawings CW-002 and 003 in plan view and sections. A typical section is shown on the figure below.
The dam is a mass concrete retaining structure, resisting horizontal forces mainly due to its own weight. Horizontal forces on the dam consist essentially of water pressure due to differences in water levels on both sides of the wall, and seismic forces.
The core of the dam is roller compacted concrete, covered on both sides with a conventional reinforced concrete layer.
The width of the structure at the footing is approximately 19m, while the height in the largest section reaches 23.50m (from +7.00m PLD till +30.50m PLD).
Beneath the footing there is a grout curtain to avoid seepage, as well as an additional consolidation grouting which can be minimized in function of the rock condition after excavation.



Rio Cocoli Dam - Typical cross section

B. Stability Analysis

As already indicated, the loadings on the dam consist mainly of its self weight, the water pressure and the seismic forces.

The self weight can easily be derived from the geometrical shape of the structure as shown above, and the volumetric weight of the concrete.

The water pressure acting on the structure is derived from the extreme water levels on both sides :

- minimum level downstream : +23.90m PLD;
- maximum level upstream : +30.68m PLD;

being the minimum Gatun Lake level and the level corresponding with the probable maximum flood respectively.

Finally, as far as the seismic forces are concerned, it will be assumed that $k_e = 0.21g$, corresponding with the assumptions retained for the lock structure stability analysis, which is rather severe since it may be assumed that a collapse of the dam cannot lead to catastrophic consequences.

Stability calculations are given in Appendix 5, both for extreme operating conditions as during earthquake.
It is shown that all requirements (overturning, sliding, concrete pressure, rock pressure) are met in extreme operating conditions.
During earthquake (0.21g), it was found that it is necessary to fill the excavation on both sides of the dam with rock fill in order to meet stability requirements corresponding to this load case.

3 Lower Pond

The lower pond will be situated on the west bank of the new bypass canal as shown on drawing CW-001. This location has been retained after analyzing available areas, site inspection and optimization of the recycling system layout.
The chosen location coincides partly with a dredged sludge disposal site, partly filled already with material originating from the Pacific entrance to the existing locks. The picture below shows this location (June 2003).

For detailed design purposes (excavation, slope stability,…) it will be necessary to undertake some additional field work, such as detailed topographical survey, geo-technical site investigation (borings, penetration tests), especially to determine the exact level of the sound rock. At this moment we only dispose of the results of some geological borings in the neighborhood, showing that the sound rock is situated below level –3.41m PLD and is covered with soft soil. It will be assumed for conceptual design purposes that the rock is situated at –4.00m PLD.

According to the hydraulic design, the water level in the lower pond will fluctuate in operating conditions between –7.70m PLD and –3.29m PLD, taking into account a net surface of 240,000m². The bottom of the reservoir is situated at –8.50m PLD (highest level), with a gentle slope in the direction of the pumping station with a deeper cut at level –13.00m PLD.

A typical cross section over the side slope of the pond is shown on the figure below.

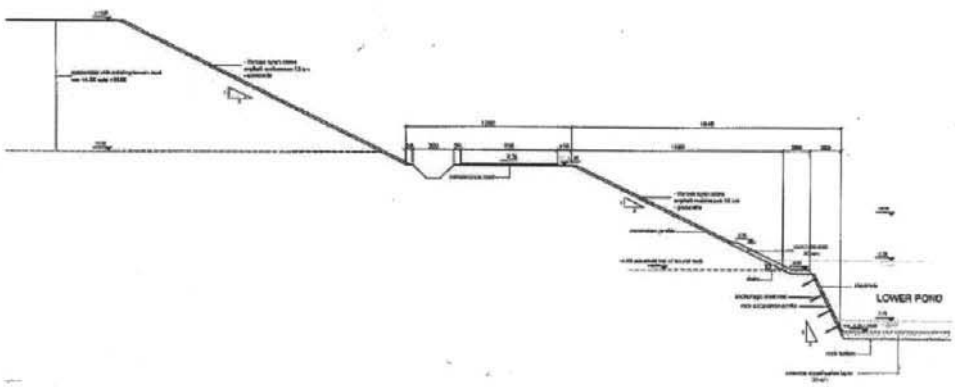


Figure :Lower Pond – Side Slope Layout

The side slopes have been designed at 1:2 (H:V) in rock below level –4.00m PLD, while an inclination of 2:1 (H:V) has been adopted for the overburden soft layers. It is understood that these inclinations will have to be verified on basis of geo-technical sampling and testing for detailed design purposes.

The lower part of the top slope (2:1), between levels –2.00m and –4.00m PLD in soft soil, will be protected by means of a reinforced concrete layer (watertight), while the upper part of the top slope needs erosion protection which can consist of different solutions. In this case (conceptual design) a slope protection consisting of a fibrous open stone asphalt layer on a geo-textile has been foreseen. The protection has to be permeable to avoid upward water pressures induced by higher ground water levels behind the slope.

The bottom of the reservoir (in rock) will be covered with a concrete equalization layer in an average thickness of 0.5m.

The side slopes in rock will also be protected with a thin shotcrete layer, armored with a steel reinforcement mesh, and anchored in rock by means of grouted rock anchors.

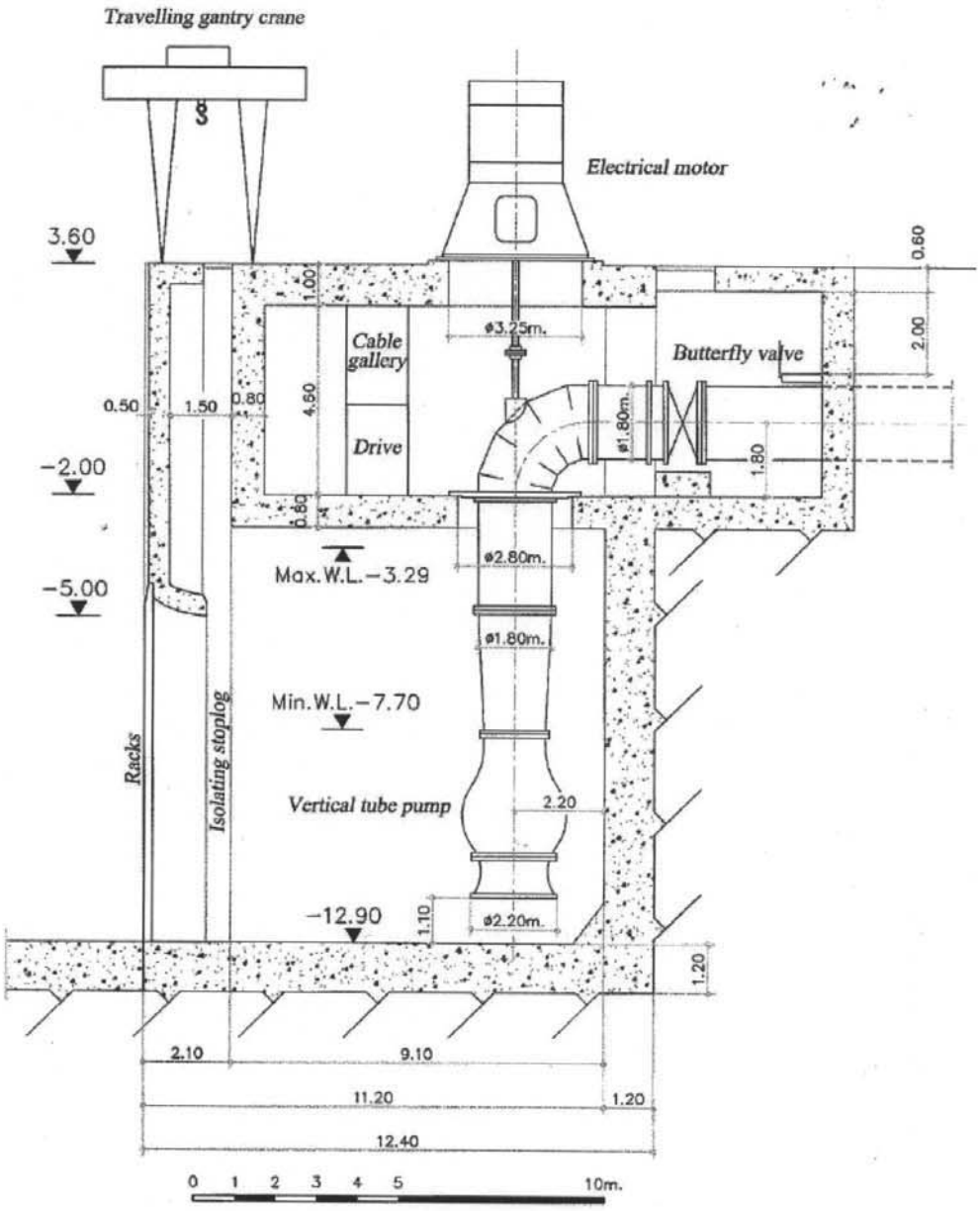
4 Pumping Stations

4.1 PUMPING STATION LOWER POND

4.1.1 GENERAL LAYOUT

The general layout of the pumping station in the lower pond is shown on drawing CW-010. A typical cross-section is given on the figure below. This layout is the same for the two base line scenarios 2 and 3 (semi direct pumping and pond to pond pumping).

Electro-mechanical installations and corresponding design criteria are given in report R-EM-02.
It should be mentioned that the pumping station dimensions may be different according to the different recycling scenarios. Indeed, the pumping station can either be designed for 3 pumps (scenario with 1-5 ship transits), 6 pumps (scenario with 5 – 10 ship transits), or 9 pumps (10 – 15 transits). However, once that the lower pond and the recycling system are in operation, it is practically impossible to extend the pumping station with additional compartments without closing off the recycling system, thus allowing for the corresponding construction works. Moreover, these extension works will be more expensive and more complicated due to the existing pumping structure. It will therefore be assumed that the pumping station will be constructed for the maximum operating conditions, it means with the maximum required number of pump compartments, a power supply room, and a maintenance platform.
At the other hand, the pumps and other related machinery need only to be installed in function of required capacities corresponding to lock operation demands.
The pumping station structure can be compared to a retaining wall consisting of a vertical wall with counter forts and a horizontal footing. The general stability analysis is given in the next chapter.


Travelling gantry crane
Electrical motor
3.60
0.60
1.00
ø3.25m.
Cable gallery
Butterfly valve
2.00
0.50
1.50
0.80
4.50
ø1.80m.
Drive
1.80
-2.00
0.80
ø2.80m.
Max.W.L.-3.29
-5.00
ø1.80m.
Min.W.L.-7.70
Racks
Isolating stoplog
Vertical tube pump
2.20
-12.90
1.10
ø2.20m.
1.20
2.10
9.10
11.20
1.20
12.40
0 1 2 3 4 5 10m.

4.1.2 STABILITY ANALYSIS

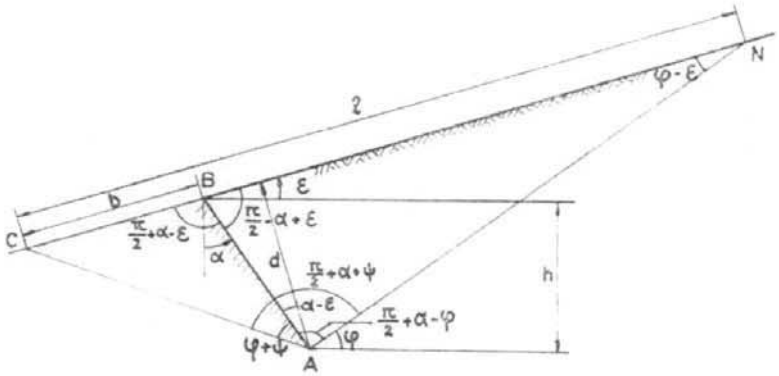
4.1.2.1 Loads

A. Self Weight (case 1)

Concrete $\gamma = 24.5$ kN/m³
Wet Soil $\gamma = 20.0$ kN/m³
Dry Soil $\gamma = 18.0$ kN/m³

B. Earth Pressure (case 2)

The active earth pressure will be calculated using the formula of COULOMB-PONCELET



$$\lambda_a = \frac{\cos^2(\varphi - \alpha)}{\cos^2\alpha \cos(\alpha + \psi)\left(1 + \sqrt{\frac{\sin(\varphi + \psi)\sin(\varphi - \varepsilon)}{\cos(\alpha + \psi)\cos(\varepsilon - \alpha)}}\right)^2}$$

The angle of friction in the filling of crushed stone is 45°
On the surface a load p=10 kN/m² is applied. (5 kN/m² in seismic conditions)

C. Water Pressure (case 3)

Inside the reservoir, the minimum water level will be applied.

At the backside of the wall the mean water level of the Pacific Ocean will be applied.

D. Seismic Loads MDE (case 4)

Seismic loads on concrete structure
Earth Pressure in seismic conditions

D.1 Earthquake level

According to the PIANC "Seismic Design Guidelines for port structures" two levels of earthquake motions have to be considered

Level 1 (L1): the level of earthquake motions that are likely to occur during the life – span of the structure
Level 2 (L2): the level of earthquake motions associated with non-frequent rare events. (Return period = 475 years for port structures with a life-span of 50 years)

For each level an acceptable damage level is defined.

D.2 Performance Grade

The highest performance grade (Grade S) is applicable for

- critical structures with potential for extensive loss of human life and property
- key structures that are required to be serviceable for recovery from earthquake disaster
- critical structures that, if disrupted, devastate economic and social activities

This means that the level of damages is:

- Structural: minor or no damage
- Little or no loss of serviceability

for both levels L1 and L2.

D.3 Analysis type

As this report concerns a conceptual design we will use the simplified analysis.

D.4 Equivalent seismic coefficient ke

Results for the design earthquakes:

Return periods->	**MCE** -	**MDE** 950 yrs	**OBE** 145 yrs	***SMDE*** *1000 yrs*	***SDE*** *475 yrs*
North Panama Deformed Belt	0.21	0.16	0.13	*0.16*	*0.15*
Pedro Miguel Fault	0.21	0.18	0.13	*0.18*	*0.16*
Recommended PGA (in g) ***(for rock conditions)***	**0.21**	**0.18**	**0.13**	***0.18***	***0.16***
	ICOLD			*UBC*	

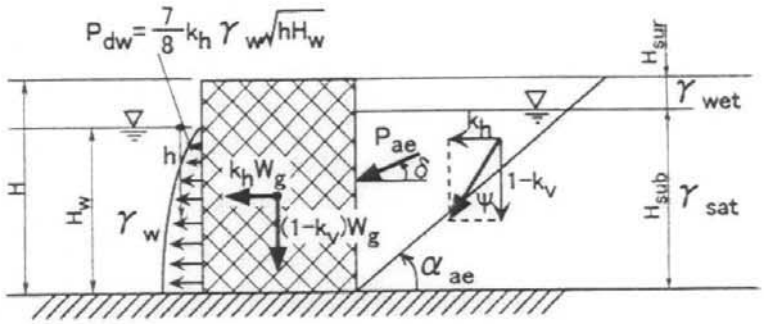
With MCE = Maximum Credible Earthquake
MDE = Maximum Design Earthquake

OBE = Operating Basis Earthquake
SMDE = Site Maximum Design Earthquake
SDE = Site Design Earthquake

MDE for L2 will be used. This means that only small deformations during MDE will be allowed.
L1 is not determining

A catastrophic failure of the pumping station structure may cause flooding of the higher dry levels where the power supply room and other equipment are installed.. That's why in addition, the calculations will be repeated for MCE, allowing higher deformations but no loss of stability.

k_e = PGA/g for PGA < 0.2 g for retaining walls
so $k_v = k_h$ = 0.18 for MDE
and $k_v = k_h$ = 0.21 for MCE



(a)

E. Valve (Case 5)

1500kN.horizontal force acting in the direction of the reservoir (emergency closure)

F. Water Pressure in Dry Conditions (case 6)

Identical as case 3 but no water pressure at the front side (maintenance condition)

4.1.2.2 Load Combinations

According to ROSA 2000

A. Quasi Permanent Load Combination

CASE101 = 1.00 x [1] + 1.00 x [2] + 1.00 x [3]

B. Accidental Load Combination - MDE

CASE102 = 1.00 x [1] + 1.00 x [3] + 1.00 x [4]

C. Rare Load Combination

CASE103 = 1.00 x [1] + 1.00 x [2] + 1.00 x [6]

D. Fundamental Load Combinations

CASE 104 = 1.20 x [1] + 1.20 x [2] + 1.00 x [3] + 1.20 x [5]
CASE105 = 0.90 x [1] + 1.20 x [2] + 1.00 x [3] + 1.20 x [5]

4.1.2.3 Criteria

According to ROSA 2000

A. Sliding along the base

It is assumed that the footing of the structure is embedded in sound rock, in such a case sliding along the base cannot occur.

B. Turn over

Load Combination	Criterion
101	Compression zone A' > 90% A
103	Compression zone A' > 75% A
102, 104 and 105	Compression zone A' > 10% A

With triangular distribution of soil pressure.

C. Load Bearing Capacity

$\gamma_d \cdot q_{ref} < q_u$
γ_d . = 1.4 for load combination 103 to 104
γ_d . = 1.0 for load combination 102 (MDE)

q_u = ultimate load bearing capacity
q_{ref} = reference pressure

D. Deformations

The deformations will be checked for frequent and rare load combinations 101,102 and 103 (MDE)

E. Concrete Stresses

Fundamental load combinations: 103 tot 104: $1.125 \times \sigma_c < 0.85\ f_{ck}/1.5$
Accidental load combination: 102 (MDE): $\sigma c < 0.85\ f_{ck}$

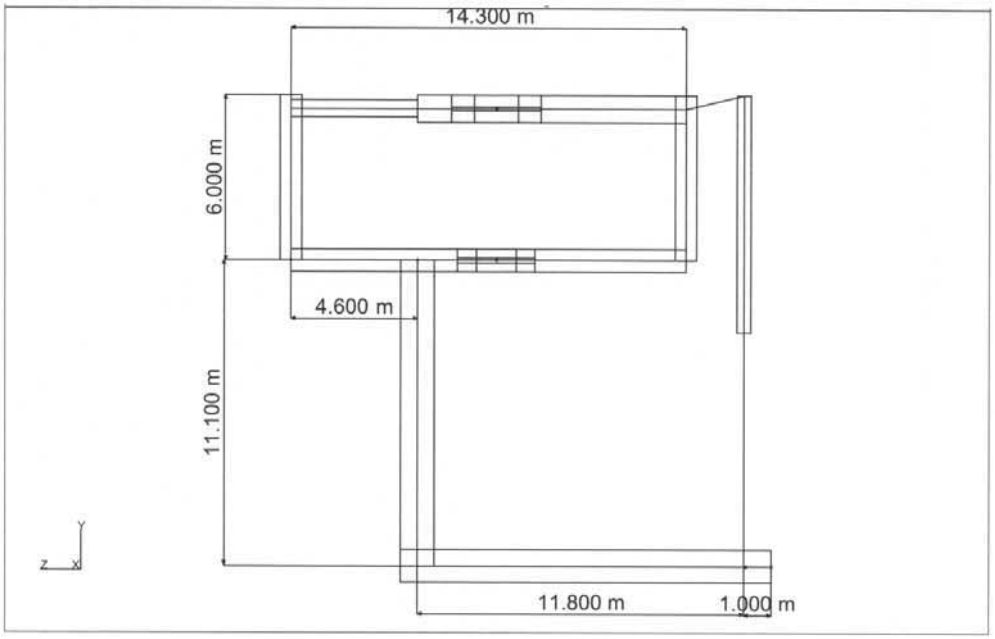
The factor 1.125 is a result of combination of the different factors in the load combinations between ROSA 2000 and Eurocode

4.1.2.4 Software

The calculations are made by means of "EFFEL", a finite element program.
The model, which will be used, is a 3D-model.

4.1.2.5 Input Data

A. Geometry



B. Materials

Material	E kN/m²	Nu	W/V kN/m3	Alpha
CONCRETE	19620000.000	0.100	24.525	0.0000100

C. Supports

The rock base is modeled by means of a surface support.

Vertical:
Linear support with K= 1000000kN/m² is introduced.
This support only works when compressed.
Horizontal:
Linear support with K= 1000000kN/m² is introduced.
This support only works when compressed.
The K-values are estimated values.

D. Loads

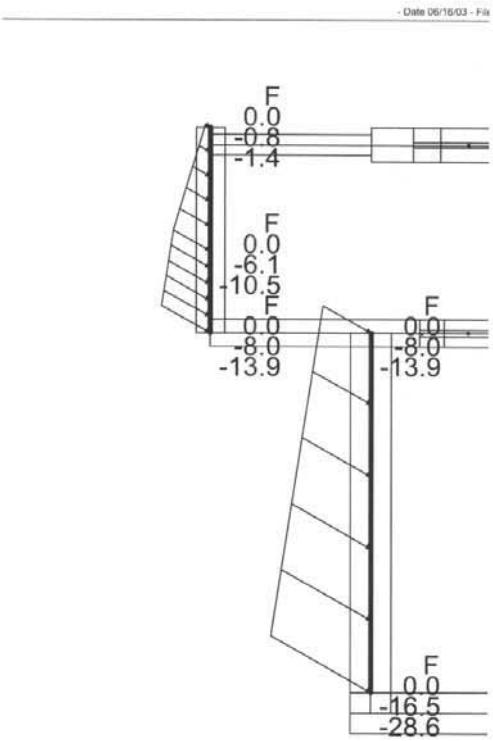
D.1 Self Weight (case 1)

D.2 Earth Pressure (case 2)

from	3.6		0	
			λav =	λah =
λa =	0.1620		0.0810	0.1403
φ =	45		δ =	30
α =	0		α+δ =	30
z	p		**pv**	**ph**
3.6	10		0.810	1.403
0	74.8		6.057	10.491

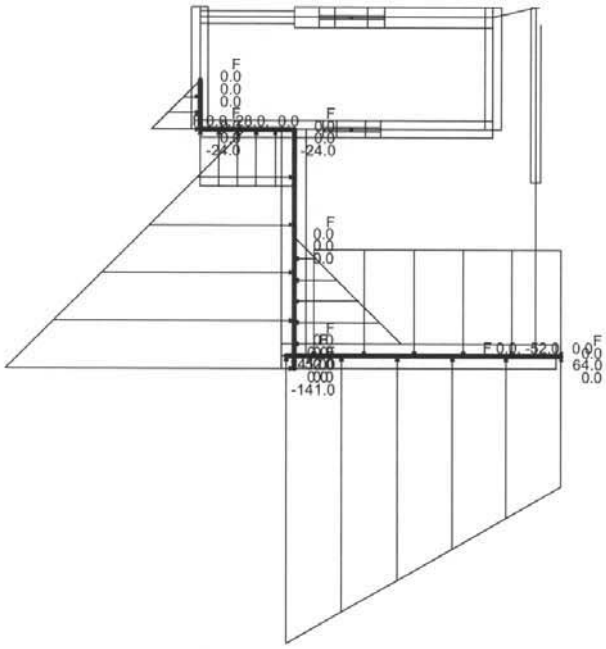
from	0	to	-2.4	
			λav =	λah =
λa =	0.1620		0.0810	0.1403
φ =	45		δ =	30
α =	0		α+δ =	30
z	p		**pv**	**ph**
0	74.8		6.057	10.491
−2.4	98.8		8.001	13.858

from	-2.4	to	-12.9	
			λav =	λah =
λa =	0.1620		0.0810	0.1403
φ =	45		δ =	30
α =	0		α+δ =	30
z	p		**pv**	**ph**
−2.4	98.8		8.001	13.858
−12.9	203.8		16.503	28.585



D.3 Water Pressure (case 3)

Water level inside: -7.7 m
Water level outside: +0.00m



D.4 Seismic Loads

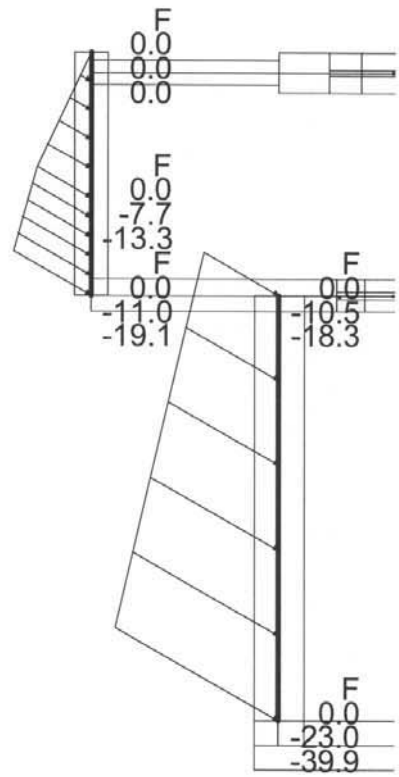
According to the PIANC regulations for Seismic design of Port Structures with $k_v = k_h = 0.18$ for MDE

Earth Pressure in seismic conditions

γd =	18			3.6
γn =	20			14.1
γn-γw =	10			17.7
γe =	12.92			
MDE	PGA=	0.18		
	ke=	0.18		
kv=	0.18			
kh=	0.18	k'h=	0.27	
ψ =	18.1			

WALL

from	3.6 to	0	
		λav =	λah =
λa =	0.2892	0.1446	0.250454
ε=	18.1		
ϕ =	45		
α=	18.1		
δ =	30		
α+δ−ε =	30		
		pv	ph
3.6	0	0	0
0	53.136	7.683459	13.30814
from	0 to	-2.4	
		λav =	λah =
λa =	0.2892	0.1446	0.250454
ε=	18.12337		
ϕ =	45		
α=	18.1		
δ =	30.0		
α+δ−ε =	30		
		pv	ph
0	53.136	7.683459	13.30814
−2.4	72.816	10.52918	18.23708
from	-2.4 to	-12.9	
		λav =	λah =
λa =	0.2892	0.1446	0.250454
ε=	18.12337		
ϕ =	45		
α=	18.1		
δ =	30.0		
α+δ−ε =	30		
		pv	ph
−2.4	72.816	10.52918	18.23708
−12.9	158.916	22.97923	39.8012



D.5 Valve

A horizontal load of 1500kN will be applied

D.6 Water pressure – dry conditions

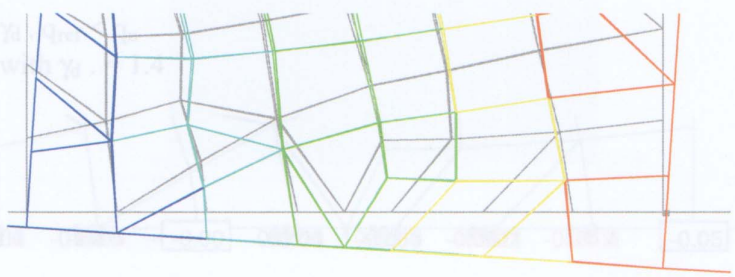
Water level inside: -12.9 m
Water level outside: +0.00m

(Same as 3 but without water inside the reservoir)

4.1.2.6 Turn over

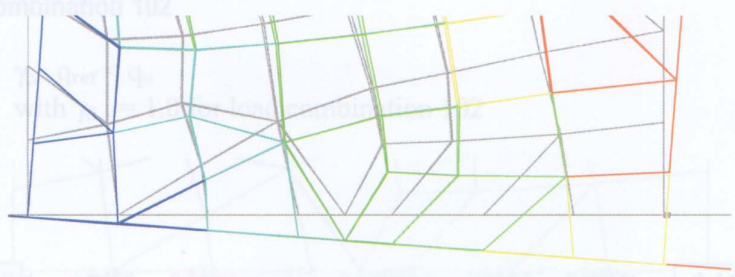
A. Quasi Permanent Load Combination 101 – Lock in service

The compression zone is the zone with negative displacements
Criterion: Compression zone A' > 90% A



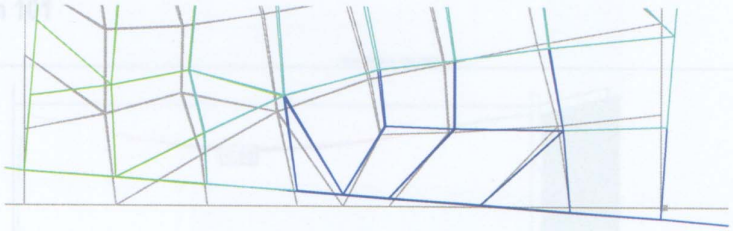
B. Rare Load Combination 103

Criterion: Compression zone A' > 70% A



C. Load Combinations 102, 104 and 105

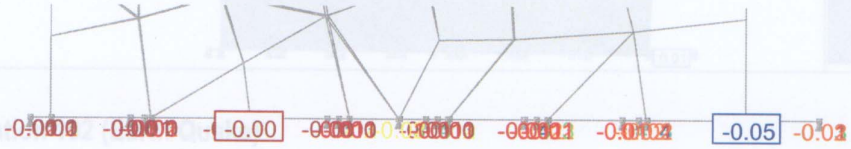
Criterion: Compression zone A' > 10% A



4.1.2.7 Load Bearing Capacity

A. Fundamental Load Combinations 104 to 105

Criterion: $\gamma_d \cdot q_{ref} < q_u$
with γ_d . = 1.4



The maximal vertical displacement at the bottom of the wall is 0.05 cm
This corresponds with a vertical pressure of 0.0005 x 1000000 kN/m² = 500 kN/m² = 0.5MPa

B. Accidental load combination 102

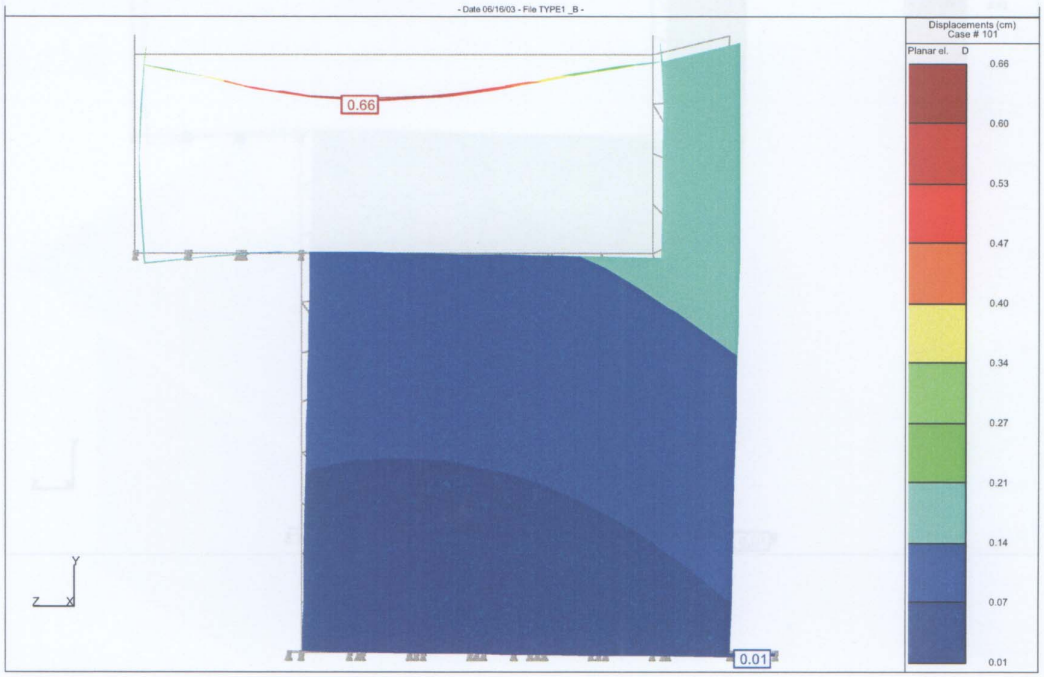
Criterion: $\gamma_d \cdot q_{ref} < q_u$
with γ_d . = 1.0 for load combination 102

The maximal vertical displacement at the bottom of the wall is 0.11 cm
This corresponds with a vertical pressure of 0.0011 x 1000000 kN/m² = 1100 kN/m² = 1.1 MPa

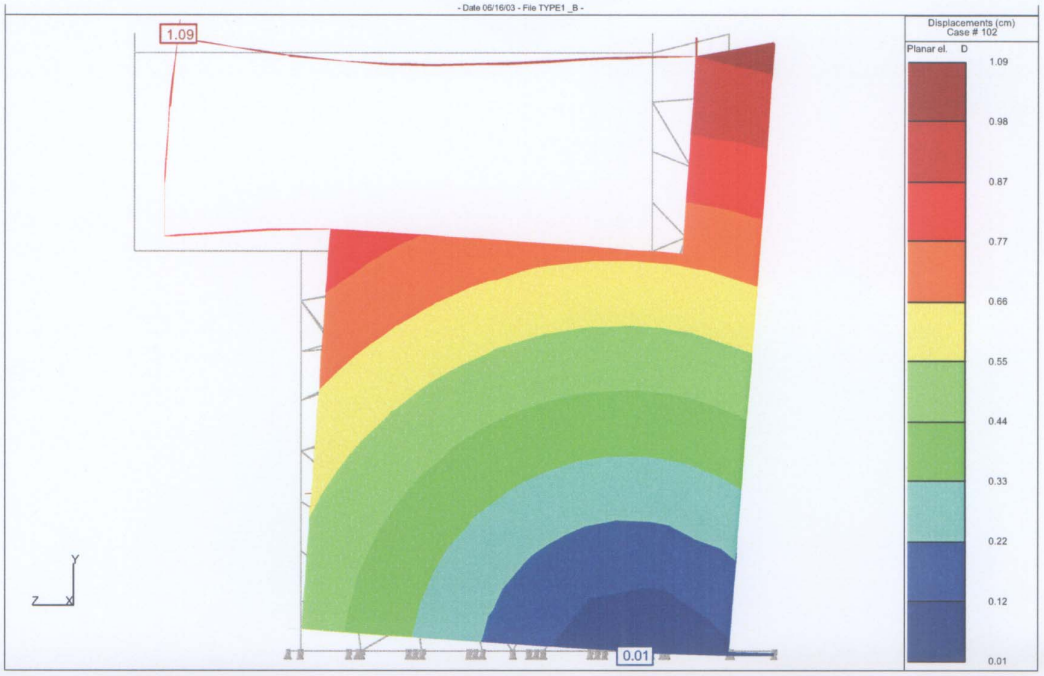
The uni-axial Compressive Strength of the basalt rock varies from 15 to 100MPa.

4.1.2.8 Deformations

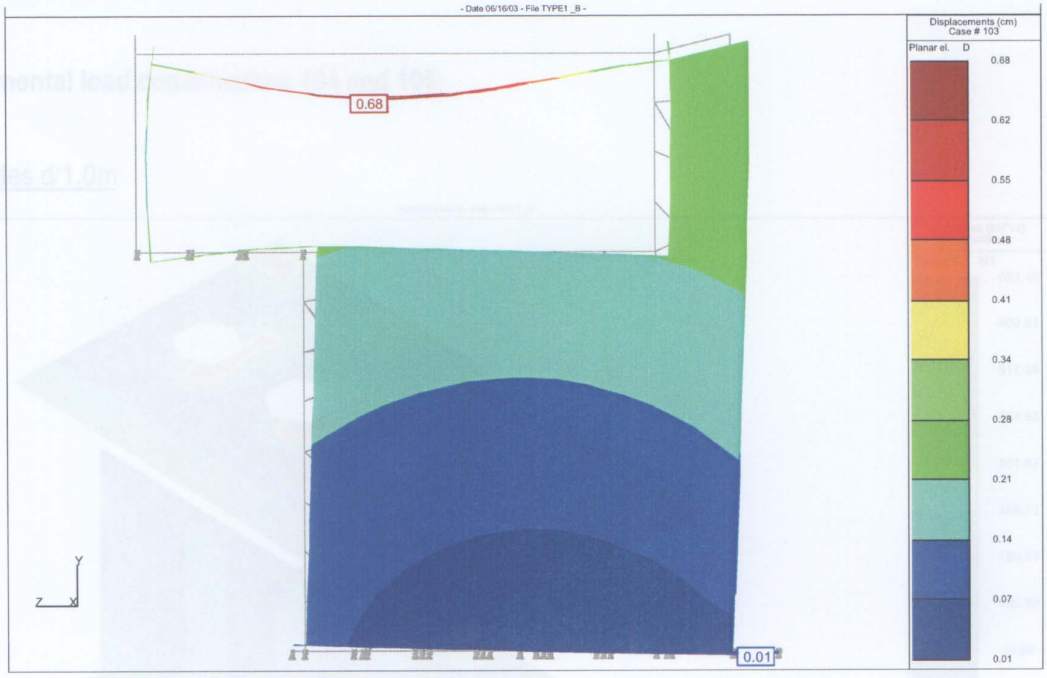
A. Load Combination 101



B. Load Combination 102 (Earth Quake)



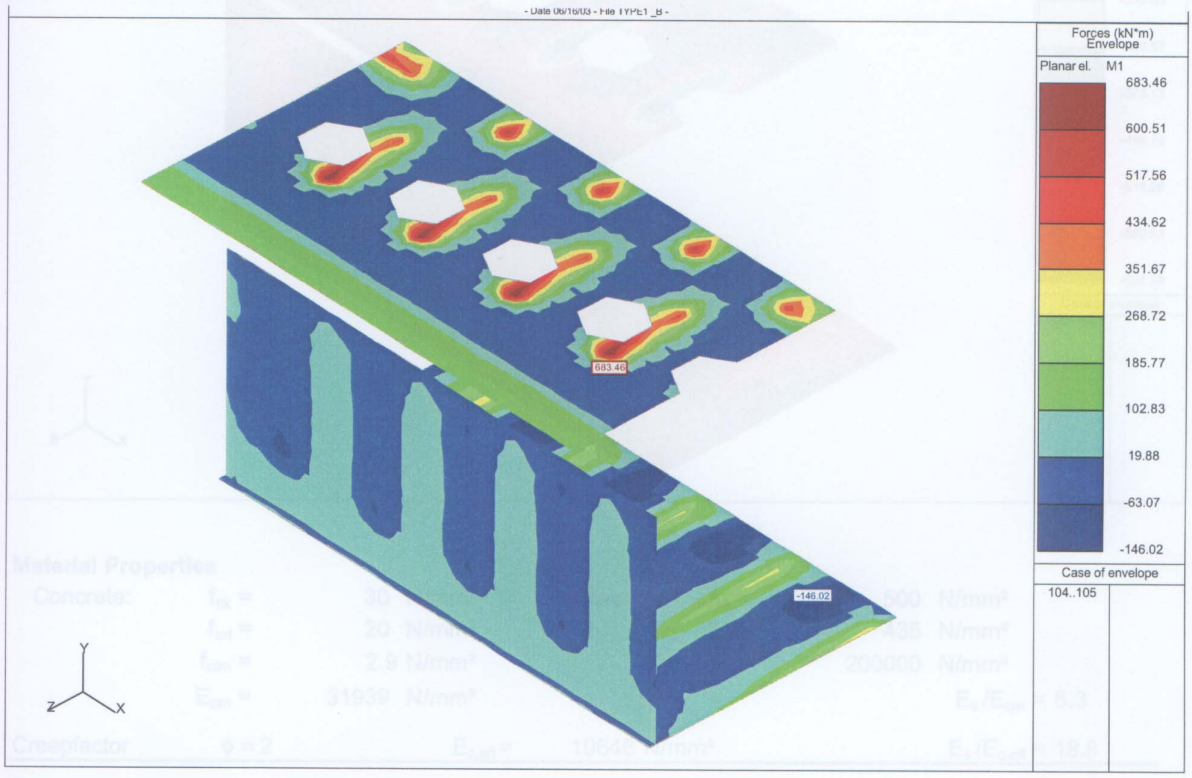
C. Load Combination 103 (Dry Conditions)

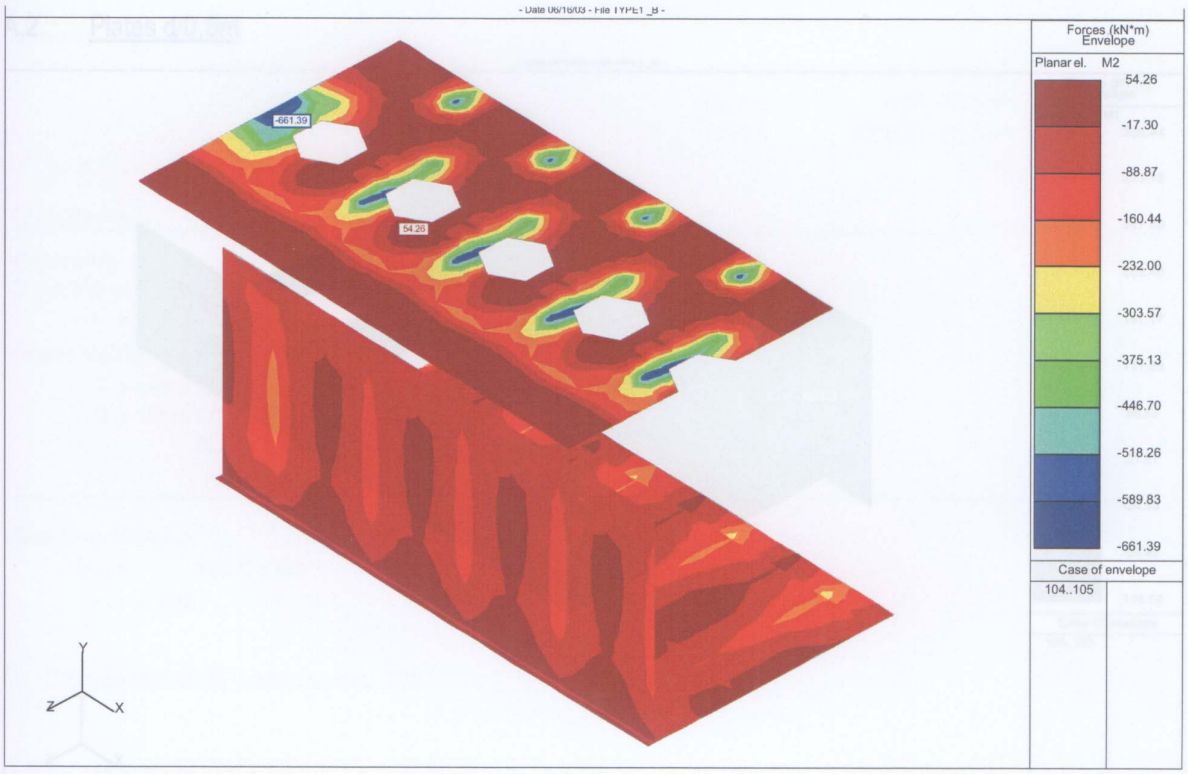


4.1.2.9 Bending Moments

A. Fundamental load combinations 104 and 105:

A.1 Plates d/1.0m





Material Properties

Concrete:	f_{ck} =	30 N/mm²	Steel:	f_{yk} =	500 N/mm²	
	f_{cd} =	20 N/mm²		f_{yd} =	435 N/mm²	
	f_{ctm} =	2.9 N/mm²		E_s =	200000 N/mm²	
	E_{cm} =	31939 N/mm²				E_s/E_{cm} = 6.3
Creepfactor	ϕ = 2		$E_{c,eff}$ =	10646 N/mm²		$E_s/E_{c,eff}$ = 18.8

Geometry

Concrete section

h = 1000 mm b = 1000 mm

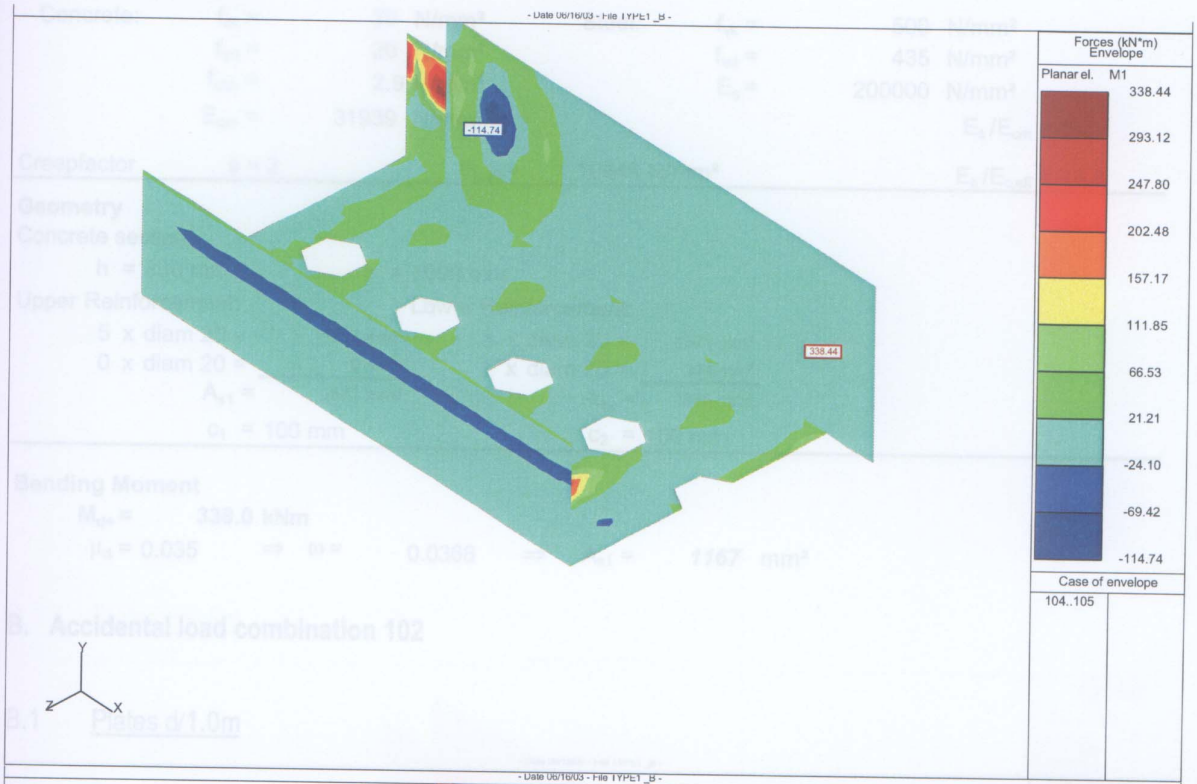
Upper Reinforcement:		Lower Reinforcement:	
6.57 x diam 20 =	2064 mm²	6.57 x diam 20 =	2064 mm²
0 x diam 20 =	0 mm²	0 x diam 20 =	0 mm²
A_{s1} =	2064 mm²	A_{s2} =	2064 mm²
c_1 = 100 mm		c_2 = 100 mm	

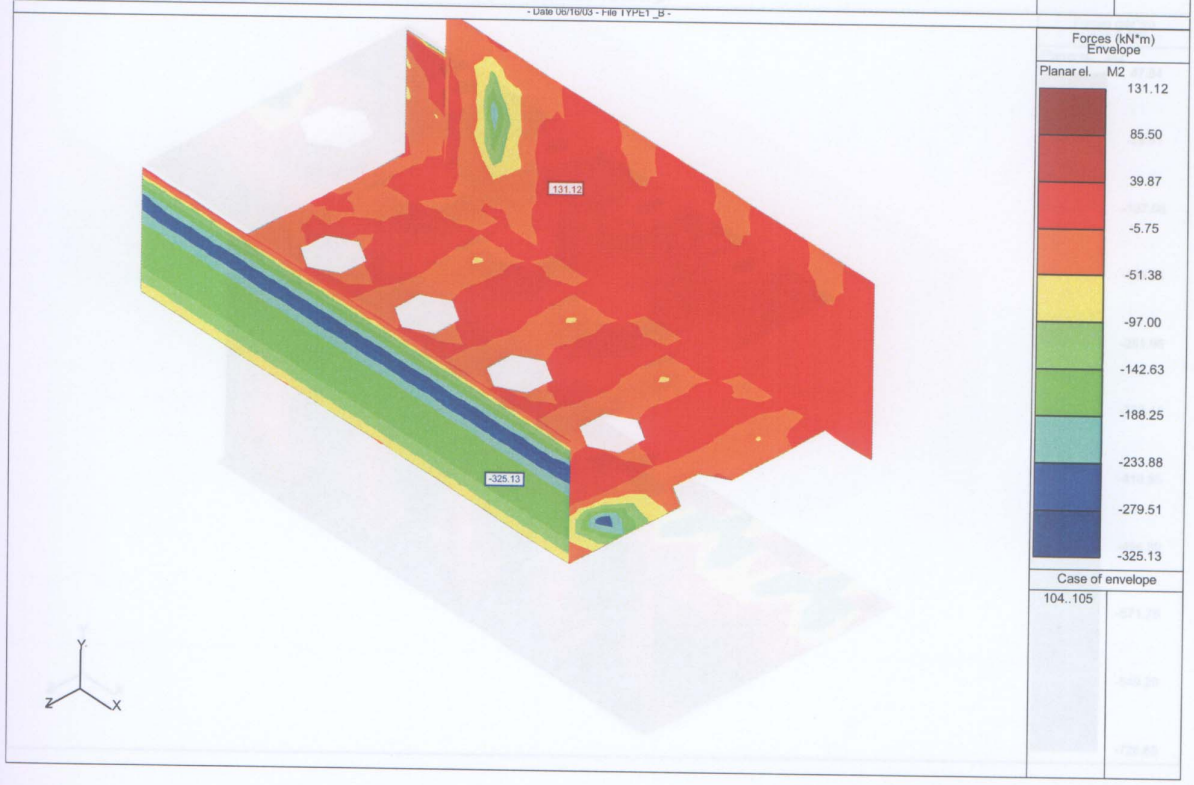
Bending Moment

M_{d+} = **700.0 kNm**

μ_d = 0.044 ⇒ ω = 0.0460 ⇒ A_{s1} = ***1883*** mm²

A.2 Plates d/0.8m





Material Properties

Concrete:	f_{ck} =	30 N/mm²	Steel:	f_{yk} =	500 N/mm²
	f_{cd} =	20 N/mm²		f_{yd} =	435 N/mm²
	f_{ctm} =	2.9 N/mm²		E_s =	200000 N/mm²
	E_{cm} =	31939 N/mm²			E_s/E_{cm} = 6.3

Creepfactor ϕ = 2 $E_{c,eff}$ = 10646 N/mm² $E_s/E_{c,eff}$ = 18.8

Geometry

Concrete section

h = 800 mm b = 1000 mm

Upper Reinforcement:		Lower Reinforcement:	
5 x diam 20 =	1571 mm²	5 x diam 20 =	1571 mm²
0 x diam 20 =	0 mm²	1 x diam 20 =	314 mm²
A_{s1} =	1571 mm²	A_{s2} =	1885 mm²
c_1 = 100 mm		c_2 = 100 mm	

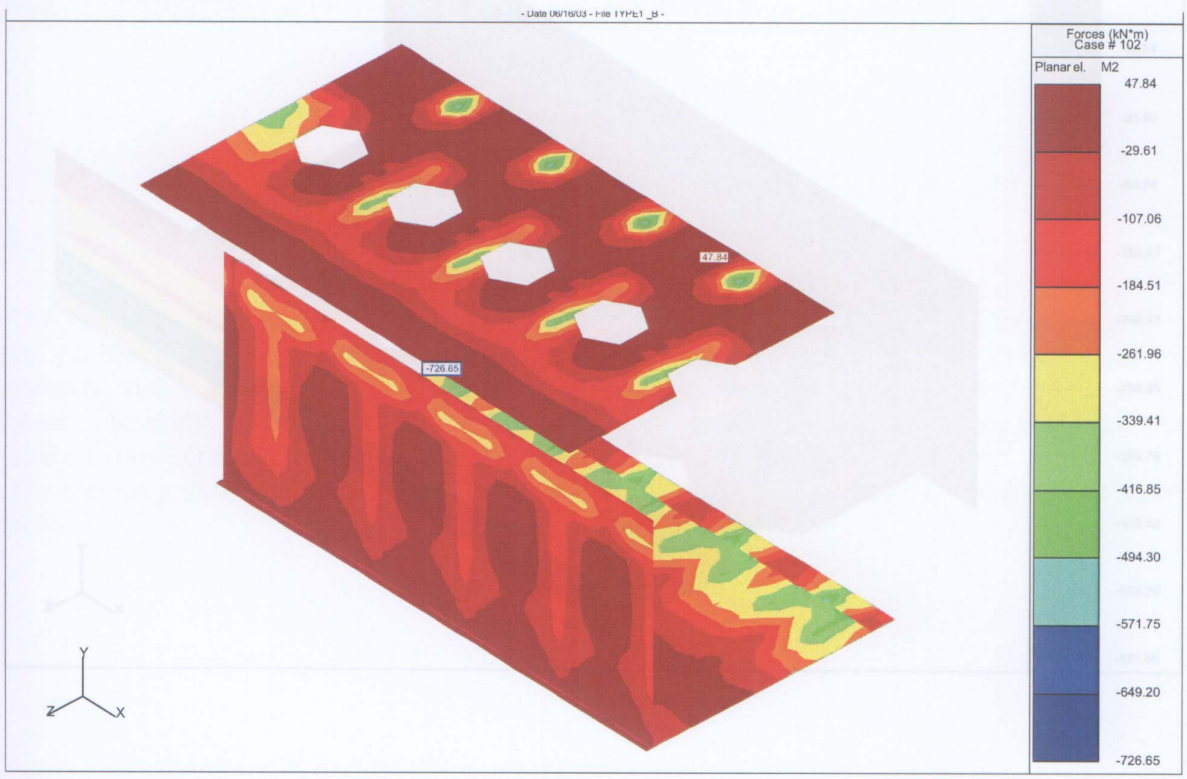
Bending Moment

M_{d+} = **338.0 kNm**

μ_d = 0.035 ⇒ ω = 0.0368 ⇒ A_{s1} = ***1167*** mm²

B. Accidental load combination 102

B.1 Plates d/1.0m



Material Properties

Concrete:	f_{ck} =	30 N/mm²	Steel:	f_{yk} =	500 N/mm²
	f_{cd} =	23 N/mm²		f_{yd} =	500 N/mm²
	f_{ctm} =	2.9 N/mm²		E_s =	200000 N/mm²
	E_{cm} =	31939 N/mm²			E_s/E_{cm} = 6.3

Creepfactor $\phi = 2$ $E_{c,eff}$ = 10646 N/mm² $E_s/E_{c,eff}$ = 18.8

Geometry

Concrete section

h = 1000 mm b = 1000 mm

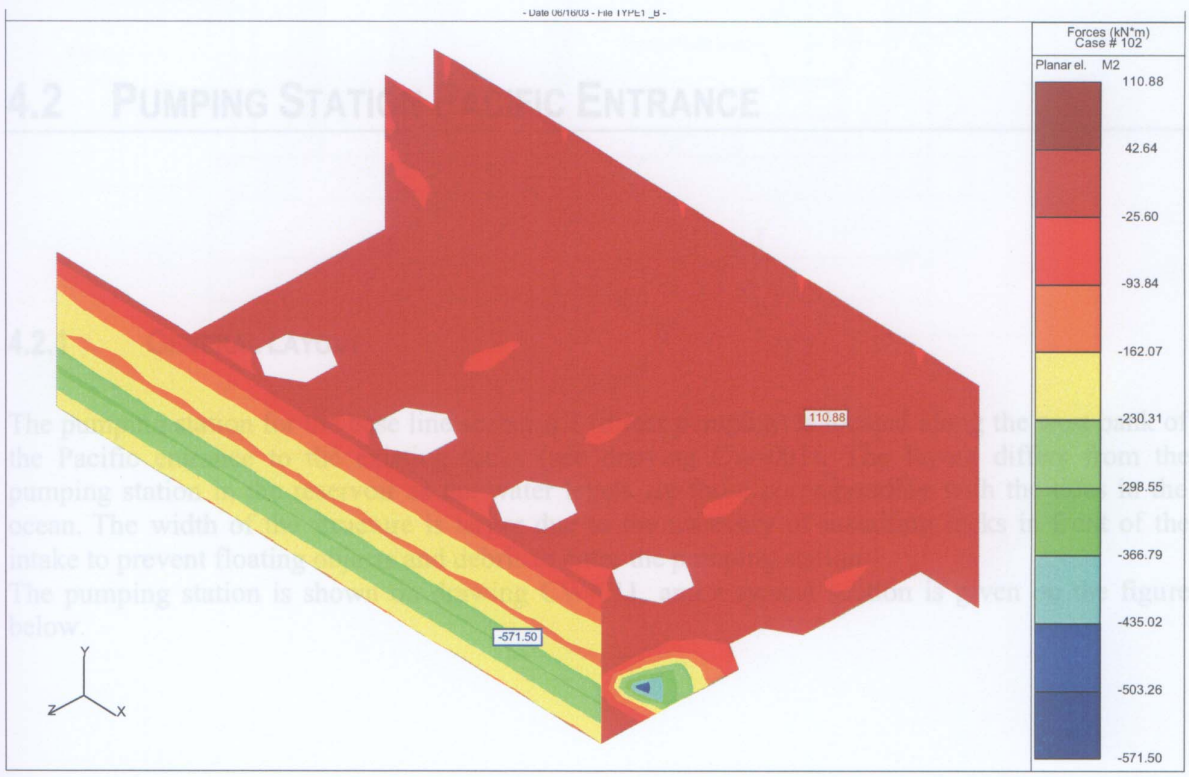
Upper Reinforcement:		Lower Reinforcement:	
6.67 x diam 20 =	2095 mm²	5 x diam 20 =	1571 mm²
0 x diam 20 =	0 mm²	0 x diam 20 =	0 mm²
A_{s1} =	2095 mm²	A_{s2} =	1571 mm²
c_1 = 100 mm		c_2 = 100 mm	

Bending Moment

M_{d+} = 727.0 kNm

μ_d = 0.040 ⇒ ω = 0.0413 ⇒ A_{s1} = ***1696*** mm²

B.2 Plates d/0.8m



Material Properties

Concrete:	f_{ck} =	30 N/mm²	Steel:	f_{yk} =	500 N/mm²
	f_{cd} =	23 N/mm²		f_{yd} =	500 N/mm²
	f_{ctm} =	2.9 N/mm²		E_s =	200000 N/mm²
	E_{cm} =	31939 N/mm²			E_s/E_{cm} = 6.3
Creepfactor	ϕ = 2		$E_{c,eff}$ =	10646 N/mm²	$E_s/E_{c,eff}$ = 18.8

Geometry

Concrete section

h = 800 mm b = 1000 mm

Upper Reinforcement:		Lower Reinforcement:	
6.67 x diam 20 =	2095 mm²	5 x diam 20 =	1571 mm²
0 x diam 20 =	0 mm²	0 x diam 20 =	0 mm²
A_{s1} =	2095 mm²	A_{s2} =	1571 mm²
c_1 = 100 mm		c_2 = 100 mm	

Bending Moment

M_{d+} = **571.0 kNm**

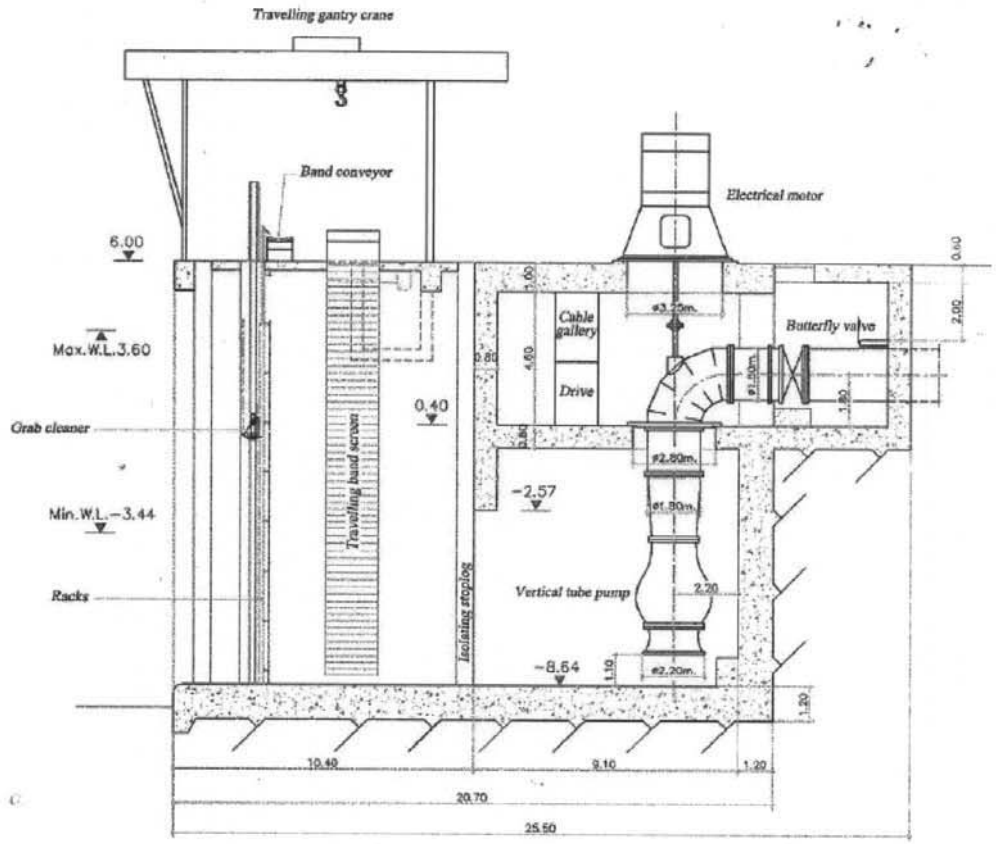
μ_d = 0.052 ⇒ ω = 0.0543 ⇒ A_{s1} = ***1730*** mm²

4.2 PUMPING STATION PACIFIC ENTRANCE

4.2.1 GENERAL LAYOUT

The pumping station for the base line scenario 1 (direct pumping) is located along the west bank of the Pacific entrance to the existing locks (see drawing CW-001). The layout differs from the pumping station in the reservoir as the water levels are those corresponding with the tides in the ocean. The width of the structure is larger due to the necessity of installing racks in front of the intake to prevent floating objects and debris to enter the pumping station.

The pumping station is shown on drawing CW-011, and a typical section is given on the figure below.



SECTIONAL VIEW

0 1 2 3 4 5 10m.

4.2.2 STABILITY ANALYSIS

4.2.2.1 General Stability

- The difference in water pressure between inside and outside is only 3.44 m instead of 7.7
- The width of the construction is 20.7 in stead of 12.4m
- The height of the construction is 15.8m in stead of 17.7m

We can conclude that general stability will be no problem as in general the situation is more favorable than those for the pumping station in the lower pond.

4.2.2.2 Load Bearing Capacity

The vertical pressures will be smaller than those found in the analysis of the Lower Pond Pumping Station.

4.2.2.3 Concrete Stresses

The bending moments will be of the same order of magnitude as those found in the analysis of the Lower Pond Pumping Station.

4.2.2.4 Deformations

Deformations depend mainly on the characteristics of the rock or the subsoil beneath the structure. In this case the pumping station is situated in the La Boca formation, this rock formation is softer than the Basalt formation, and accordingly it can be expected that the deformations of a structure comparable with a structure built in Basalt will be higher. The deformation modulus of La Boca varies between 2,5 and 6,6Gpa, while Basalt is characterized by a deformation modulus of 8 up to 12,5Gpa. As the maximum vertical deformation found for the structure in Basalt is 0,11cm, the deformation of the larger structure in La Boca will not exceed 5x0,11cm = 0,55cm, which is still a very low and acceptable value.

5 Penstock

5.1 GENERAL CONSIDERATIONS

The layout of the penstock is different for each base line scenario, it is shown for each scenario on drawing CW-001.

- scenario 1 (direct pumping) : starting on the west bank of the Pacific entrance and discharging in the new bypass canal at the upstream east side of the new locks. Total length : 2400m
- scenario 2 (semi direct pumping) : starting in the lower pond and discharging on the west bank of the new locks, in the lake created to intercept the Rio Cocoli river during the construction of the new locks. Total length : 2150m
- scenario 3 (pond to pond pumping) : starting in the lower pond and ending in the upper pond (Rio Cocoli reservoir). Total length : 2520m

For each penstock a longitudinal profile has been chosen in order to minimize excavation work. Both longitudinal profiles and typical cross sections are shown on drawings CW-007/008/009. The corresponding excavation volumes have been calculated with the software AutoDesk Land Desktop.

For each baseline scenario different combinations of lock operation modes have to be considered, and for each of them corresponding penstock dimensions have been determined in the hydraulic study (report R-HY-002). Following table shows the different combinations with the resulting penstock diameters :

	number of ship transfers		
Lock configuration	1 to 5ships	5 to 10ships	10 to 15ships
3 lift + WSB	1 D1800mm	2 D1800	3 D1800
3lift without WSB	1 D2900	2 D2900	3 D2900
2lift + WSB	1 D2300	2 D2300	3 D2300
2lift without WSB	1 D3200	2 D3200	3 D3200

5.2 PENSTOCK DESIGN

Some practical considerations need to be taken into consideration when designing the penstock.

- the roughness of the inside pipe surface should be corresponding with the recommendations of the hydraulic study : it was found however that a variation of the roughness from 0,025mm to 0,3mm does not very much affect the pumping head. Therefore it can be concluded that either steel or concrete can be used as pipe material.
- the climate in Panama is warm and humid and consequently steel pipes will need to be protected against corrosion, either by painting with specific system products or by increasing the wall thickness.
- also due to the climate and exposure to the sun it can be expected that the penstock will have to be fitted with sufficient expansion joints.
- steel for pipe manufacturing will need to be imported as there are no steel plants in Panama
- concrete pipes could either be prefabricated (circular) or cast in place (rectangular)
- steel pipes are easier to replace but will need more maintenance than concrete pipes.
- cast in place concrete culverts will need much more installation time than prefabricated pipes and steel pipes
- some advantage could be taken with cast in place culverts when constructing them all three together at the same time.

It should be noticed however that normally most penstocks that we know of are in steel, and it would be rather unusual to choose in this particular case for a concrete solution. This is why we will adopt steel pipes as basic option for the recycling penstock, the concrete solution can be retained as an alternative.
The penstock can be laid in the excavated trench on supports and be left uncovered. This possibility has the advantage that the pipes are easy to inspect and to maintain or repair, but at the other hand they are exposed to unfavourable weather conditions (corrosion, dilatation), and from the esthetical point of view one might prefer to cover the pipes and restore the natural environment. Another problem that arises when leaving the trench open is the fact that it has to be dewatered by an efficient drainage, otherwise it would work as an open drain itself, bringing the water from heavy rain straight down to the pumping station together with mud, which would of course be unacceptable.
Therefore it has been preferred to refill the trench, this needs a certain volume of sand-cement or lean concrete as foundation bed and sand to envelope the steel pipes; the excavated material can be used to fill the remaining part of the trench and can be spread in place.
Covering the pipes allows to weld the pipe sections together to one continuous length.

The pipes have to resist an internal water pressure corresponding with approximately 30m of water column, and an external pressure due to soil- and water pressure of maximum 10m. These loadings lead to minimum wall thickness of only a few millimetres, and therefore minimum thickness is

mainly fixed in function of transportation and manipulation of the pipes. D/200 as a maximum for wall thickness will be used for cost estimation purposes.
The pipes will be treated with a resistant coating on both sides. Normally the penstock will be permanently filled with water, but for inspection and maintenance they can be emptied by closing the stop log at the upstream side and opening the butterfly valve in the pumping station. Inspection shafts are installed at regular intervals (vertical pipes D = 1200mm) and provided with ladders and secured covers.

At the lower side of the penstock a lateral conduit branch has been provided to anticipate water hammer. The branch conduit is directed upward along the closest hill to a level of +40.00m PLD, and ends in a concrete reservoir.

6 Connection of Recycling Ponds to Lock Chambers.

6.1 General Considerations

The connecting conduit(s) between upper pond and upper lock chamber, and lower lock chamber with lower pond consist of reinforced concrete culverts with dimensions HxB = 7,5mx9m (as the longitudinal culverts of the lock chambers).
Layout and dimensions of the conduits and connections to lock culverts are shown on drawings CW-004 and 005.
The longitudinal profile of the conduits has been determined taking into account the extension of the Canal with a fourth lane, and minimum excavation. The conduits pass beneath the future water saving basins, and under the fourth lane locks. The layout is different for the double lift lock and the triple lift lock due to the different length and the shifted location of these configurations along the future canal axis. In both cases the connection with the lock culvert is situated in the middle of the lock chamber.
As the conduits have to be closed after each filling of the lock chamber, a valve chamber is required. The valve chambers have been placed as close as possible to the lock chamber. As the culverts are lying very deep (beneath the lock chamber bottom), these valve chambers need to be very deep as well, and as such it is necessary that they be built together with the lock structure. This is also valid for the conduit that is linked to the eastern lock culvert.
If the fourth lane of locks would be constructed before a recycling system has been installed, also the valve chamber and conduit section below these locks will have to be constructed at the same time. This case has not been considered in the cost estimate, and means that only the valve chamber and the conduit under the third lane of locks are taken into account.

Part of the structure, especially the part close to the Rio Cocoli dam could be situated in less resistant soil and may need additional foundation (such as piles or soil replacement). At this moment there is insufficient information on rock/soil characteristics to analyze this matter in more detail. Additional soil investigation along the conduit axis has to be performed for detailed engineering design purposes.

6.2 CONDUIT DESIGN

As the dimensions of the conduit are rather large, the construction method retained is essentially the one of a cast in place reinforced rectangular shaped tunnel in open excavation. This will certainly be valid for the partition that is integrated in the lock structure.
Another part of the conduit is situated at or near existing ground surface level and needs only minimum excavation.
An important part of the conduit is situated at great depth (65 to 70m) and requires important excavation work, even with steep cuts. These excavations will have to be undertaken in the close vicinity of the new locks that will be operating at that time. The works will therefore be disturbing to lock operations, they will be difficult and dangerous, and last but not least they will be expensive. This construction method cannot be considered as very realistic, and is only retained for cost estimation purposes.

A more reliable and realistic construction method is the one of a "drilled an blast" tunnel. The equivalent diameter of the circular section is 9,30m, and a concrete liner section of 0,60m has been taken into account together with cement mortar injections.

Dimensions (wall thickness) of rectangular sections vary according to depth and can be further optimized during detailed design. It could for example be considered to split one conduit in two by adding a vertical wall. Some parts of the conduit that are situated in the lake need to be covered sufficiently with rock to avoid floating in maintenance condition (the conduit can be closed off at the dam and at the lower pond by means of stop logs in order to empty it).
At the valve chamber each conduit is split in two in order to have two valves on it for redundancy purposes. Each valve can be dewatered by means of stop logs installed at both sides.
The valves will be operated from the locks control room that is situated close to it. Additional power supply is available at the HV room of the locks.

7 Construction Schedule

7.1 GENERAL CONSIDERATIONS

Whenever a decision is made to build a recycling system, it will be constructed after the third lane of locks has been realized and will be operating. Only the valve chambers (two valve chambers for baseline scenario 3 – pond to pond pumping, and one valve chamber for baseline scenario 2 – semi-direct pumping) and the culverts connecting the valve chambers with the lock culverts will have to be built simultaneously with the locks. This may be of some influence on the construction schedule of the lock (locally deeper excavation, complicated concrete work), but as this schedule has been set up with rather safe assumptions as far as production rates are concerned, it can be assumed that the total construction schedule will not have to be extended.

The construction of the recycling system is different for the 3 base line scenarios. It is self evident that base line scenarios 3 and 2 will take longer than scenario 1, as no ponds and connecting conduits are required for the latter. At the other hand, several elements of the recycling systems can be built simultaneously, for example the upper pond (dam) and the lower pond with pumping station can be started from the beginning of the project, and even the penstock and connecting conduits can be realized practically independently.

Based on normal production rates for excavation, concrete works, tunneling, penstock erection, installation of electro-mechanical equipment, the following total construction periods have been estimated :

- base line scenario 1 (direct pumping) : 21 months
- base line scenario 2 (semi direct pumping) : 36 months
- base line scenario 3 (pond to pond pumping) : 36 months

It should be noted that these construction periods are valid for the minimum solution (3 pumps and one penstock), as this will be the most feasible way of setting up a recycling system. If the decision should be made to immediately realize the maximum recycling capacity , the construction schedule has to be revised and will be extended accordingly.

7.2 DIRECT PUMPING

The construction schedule for this baseline scenario 1 is shown in appendix 6.1.
There are three main activities that can determine the construction schedule of this system :

- the construction of the pumping station;
- the installation of the pumps and electro-mechanical equipment;
- the erection of the penstock.

The civil engineering works are mainly excavation work and concrete structural work, the size of the structure and the concrete volumes involved are relatively small when compared to large structures. It was found that these activities are not on the critical path and can be scheduled in function of the other main activities.
The penstock has a total length of approximately 2400m, and the production rate can be estimated at 6m/day based on experience with similar projects. Consequently it will take 400 working days to complete the penstock, or 200 days if two construction teams are mobilized.
Essentially the production process of the pumps governs the electro-mechanical equipment installation work. Contract negotiation, fabrication, testing and supply at the site will require a minimum of 15 months. Installation and testing on site can be estimated at another 6 months, which brings the total construction period to a minimum of 21 months.

7.3 SEMI-DIRECT PUMPING

As compared to the Direct Pumping scenario, two additional components have to be built :

- the conduits connecting the lower lock chamber with the lower pond, including the valve chamber;
- the lower pond.

The connecting conduit, as already highlighted before, is split up in a part that is integrated in the lock structure, and another part linking the valve chamber with the lower pond. Only the second part is significant for the construction schedule. It will be assumed that the bored tunnel method will be used, with a production rate of 10m/day. The total length is approximately 2070m, and as such it will take only 207 days to complete it.
The total volume to be excavated for the lower pond have been estimated at 1,9Mio m^3 rock and 3,6Mio m^3 overburden. If an average production capacity of 100,000m^3/month in rock and 500,000m^3/month in overburden is adopted, a construction period of 26 months would be required, and additional working time is required for side slope construction and equalization concrete spreading. As a consequence the construction works for the lower pond can be estimated to take 32 months, and as the other activities take less time they can be spread over the same time span. It allows for instance to erect the penstock with only one construction team instead of two.
The resulting construction Gantt chart is shown in appendix 6.2. The total program takes 36 months.

7.4 POND TO POND PUMPING

This baseline scenario 3 is far more complicated than the scenarios 1 and 2, as there is a dam to be constructed on the river Cocoli, as well as the connecting conduits to the upper lock chamber.
It has already been explained that part of the connecting conduits has to be built together with the new locks (culverts passing beneath the lock structure and valve chamber). These construction works may slightly increase the time schedule of the locks.
The construction of the dam, which is a rather small one will not take too much time as far as the structure itself is concerned, but the preparatory works (site access, clearing of the forest, deviation of Rio Cocoli, removal of explosives) might require more time than expected.
However if the site preparation would take 12 months, which should be reasonable, still 20 months are available to realize the dam if the schedule of the lower pond is considered as determining (32 months).
The connecting culverts (upper pond – upper lock chamber and lower pond – lower lock chamber) require deep and voluminous excavations, and it appeared during cost estimation analysis that a tunneling method for the deeper parts of the conduits would be slightly more economical.
The total tunnel length is in this case nearly 4000m, and will require 400 days (20 months) working time at a production rate of 10m/day. The remaining sections to connect the conduits with the ponds are constructed in open excavation and this would take no more than 6 months. It appears that the construction of the lower pond is still the determining activity (32 months), and as such the pond to pond recycling system can be realized within the same schedule as the semi-direct pumping system.
The Gantt chart is shown in appendix 6.3.

8 Cost Estimate

8.1 General Considerations

The detailed cost estimation for the civil engineering works is based on the bill of quantities (see appendix 7) and unit prices taken from similar works and projects. The consultant has prepared graphs showing the variation of unit prices in function of quantities at the occasion of the conceptual study of the locks, and one of the partners has also gathered useful information during the feasibility study of the Rio Toabre project.

The cost estimation of the Recycling project has been prepared for the three base line scenarios, and for each scenario 4x3 operation modes have been considered (4 possible lock configurations and 3 operation modes).

In order to have a complete overview of the required investment cost, the cost estimate of the electro-mechanical equipment has been taken from report R-EM-02 and added to the civil engineering. The cost estimate of the water hammer protection devices has been taken from the hydraulic part of the study (R-HY-002-B) and added to the cost of the civil engineering work.

Cost of maintenance and exploitation are generally estimated as a percentage of the investment cost of the hydraulic system. In this case where the civil structures represent the major part of the investment cost, it is recommended to adopt 1% as an annual O+M cost.

8.2 SCENARIO 1 : DIRECT PUMPING

Component	3Lift + WSB	3Lift no WSB	2Lift + WSB	2Lift no WSB
Pumping station	$1.176.602,00	$2.941.505,00	$2.059.054,00	$2.941.505,00
Penstock (3)	$15.876.693,00	$38.557.677,00	$26.428.890,00	$59.445.933,00
Discharge structure	$449.170,00	$449.170,00	$449.170,00	$449.170,00
EM-equipment (total)	$8.240.000,00	$17.440.000,00	$12.435.000,00	$19.730.000,00
Water hammer protection	$1.287.741,00	$2.355.725,00	$1.978.455,00	$3.199.967,00
Subtotal 1 (*)	$10.141.711,67	$22.914.548,33	$16.183.639,00	$30.933.138,67
Subtotal 2 (*)	$18.584.366,33	$42.326.888,67	$29.765.061,00	$58.347.062,33
Subtotal 3 (*)	$27.030.206,00	$61.744.077,00	$43.350.569,00	$85.766.575,00
Total 1 (**)	$12.879.973,82	$29.101.476,38	$20.553.221,53	$39.285.086,11
Total 2 (**)	$23.602.145,24	$53.755.148,61	$37.801.627,47	$74.100.769,16
Total 3 (**)	$34.328.361,62	$78.414.977,79	$55.055.222,63	$108.923.550,25
M & O yearly cost 1	$128.799,74	$291.014,76	$205.532,22	$392.850,86
M & O yearly cost 2	$236.021,45	$537.551,49	$378.016,27	$741.007,69
M & O yearly cost 3	$343.283,62	$784.149,78	$550.552,23	$1.089.235,50

(*) Subtotals 1/2/3 correspond with number of lock transits/day = 1-5 / 5-10 / 10-15

(**) Total includes 15% contingencies, 7% detailed engineering and 5% administration cost

8.3 SCENARIO 2 : SEMI DIRECT PUMPING

Component	3Lift + WSB	3Lift no WSB	2Lift + WSB	2Lift no WSB
Pumping station	$823.910,00	$2.059.775,00	$1.441.842,00	$2.059.775,00
Penstock (3)	$14.445.018,00	$34.743.897,00	$23.874.999,00	$43.447.365,00
Discharge structure	$196.980,00	$196.980,00	$196.980,00	$196.980,00
Lower Pond	$47.157.643,00	$47.157.643,00	$47.157.643,00	$47.157.643,00
Connecting culvert to lower lock chamber	$53.210.275,00	$53.210.275,00	$62.860.275,00	$62.860.275,00
Culvert and valve chamber integrated in lock (lower)	$14.758.500,00	$14.758.500,00	$14.758.500,00	$14.758.500,00
EM-equipment PS	$7.760.000,00	$15.205.000,00	$10.613.000,00	$18.540.000,00
Water hammer protection	$968.215,00	$1.882.291,00	$1.573.379,00	$2.653.951,00
Subtotal 1 (*)	$123.900.718,67	$134.705.125,33	$138.473.171,67	$148.631.991,00
Subtotal 2 (*)	$131.609.663,33	$151.958.246,67	$150.473.646,33	$170.151.516,00
Subtotal 3 (*)	$139.320.541,00	$169.214.361,00	$162.476.618,00	$191.674.489,00
Total 1 (**)	$157.353.912,71	$171.075.509,17	$175.860.928,02	$188.762.628,57
Total 2 (**)	$167.144.272,43	$192.986.973,27	$191.101.530,84	$216.092.425,32
Total 3 (°*)	$176.937.087,07	$214.902.238,47	$206.345.304,86	$243.426.601,03
M & 0 yearly cost 1	$1.573.539,13	$1.710.755,09	$1.758.609,28	$1.887.626,29
M & 0 yearly cost 2	$1.671.442,72	$1.929.869,73	$1.911.015,31	$2.160.924,25
M & 0 yearly cost 3	$1.769.370,87	$2.149.022,38	$2.063.453,05	$2.434.266,01

(*) Subtotals 1/2/3 correspond with number of lock transits/day = 1-5 / 5-10 / 10-15

(**) Total includes 15% contingencies, 7% detailed engineering and 5% administration cost

8.4 SCENARIO 3 : POND TO POND PUMPING

Component	3Lift + WSB	3Lift no WSB	2Lift + WSB	2Lift no WSB
Pumping station	$823.910,00	$2.059.775,00	$1.441.842,00	$2.059.775,00
Penstock (3)	$15.948.636,00	$38.409.402,00	$26.382.864,00	$50.856.828,00
Discharge structure	$447.440,00	$447.440,00	$447.440,00	$447.440,00
Lower Pond	$47.157.643,00	$47.157.643,00	$47.157.643,00	$47.157.643,00
Rio Cocoli Reservoir	$13.437.975,00	$13.437.975,00	$13.437.975,00	$13.437.975,00
Connecting culvert to lower lock chamber	$53.210.275,00	$53.210.275,00	$62.860.275,00	$62.860.275,00
Culvert and valve chamber integrated in lock (lower)	$14.758.500,00	$14.758.500,00	$14.758.500,00	$14.758.500,00
Connecting culvert to upper lock chamber	$47.000.000,00	$47.000.000,00	$56.250.000,00	$56.250.000,00
Culvert and valve chamber integrated in lock (upper)	$14.782.164,00	$14.782.164,00	$14.782.164,00	$14.782.164,00
EM-equipment PS	$7.700.000,00	$15.235.000,00	$10.613.000,00	$19.610.000,00
Water hammer protection	$999.960,00	$1.912.890,00	$1.621.575,00	$2.733.394,00
Subtotal 1 (*)	$199.864.972,67	$211.419.559,33	$224.048.625,67	$236.210.230,67
Subtotal 2 (*)	$208.064.708,33	$229.913.755,67	$236.899.608,33	$260.580.233,33
Subtotal 3 (*)	$216.266.503,00	$248.411.064,00	$249.753.278,00	$284.953.994,00
Total 1 (**)	$253.828.515,29	$268.502.840,35	$284.541.754,60	$299.986.992,95
Total 2 (**)	$264.242.179,58	$291.990.469,70	$300.862.502,58	$330.936.896,33
Total 3 (**)	$274.658.458,81	$315.482.051,28	$317.186.663,06	$361.891.572,38

M & O yearly cost 1	$2.538.285,15	$2.685.028,40	$2.845.417,55	$2.999.869,93
M & O yearly cost 2	$2.642.421,80	$2.919.904,70	$3.008.625,03	$3.309.368,96
M & O yearly cost 3	$2.746.584,59	$3.154.820,51	$3.171.866,63	$3.618.915,72

(*) Subtotals 1/2/3 correspond with number of lock transits/day = 1-5 / 5-10 / 10-15

(**) Total includes 15% contingencies, 7% detailed engineering and 5% administration cost

9 Appendixes

9.1 APPENDIX 1 : RIO COCOLI FLOOD HYDROGRAPHS

Datos de entrada

Long cauce=	10,2	km.
Cota max=	257	m
Cota min=	15	m
Superficie=	27,7	km2
Precipitación=	66,04	mm
Duración P neta=	1,68	horas

Cálculos

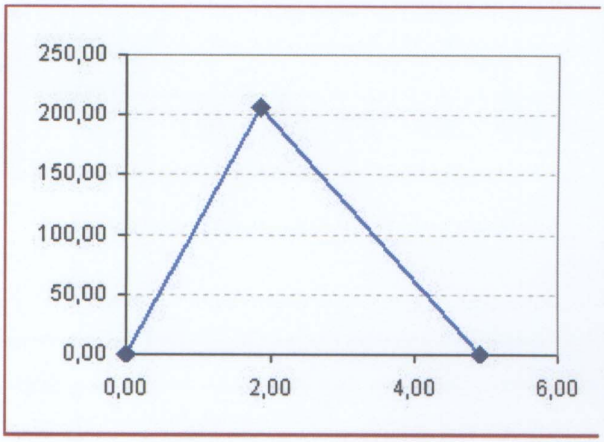
Pendiente=	0,0237 m/m
t conc=	100,2 minutos
t conc=	1,67 horas
tiempo punta=	**1,84** horas
tiempo base=	4,92 horas
Caudal de la punta=	**206,54** m^3/seg.

Q20

Datos para dibujar el triángulo

tiempo	Q
0,00	0,00
1,84	206,54
4,92	0,00

Abajo puede verse la "suavización" del hidrograma triangular por el método del Hidrograma Adimensional del SCS



Comprobación:

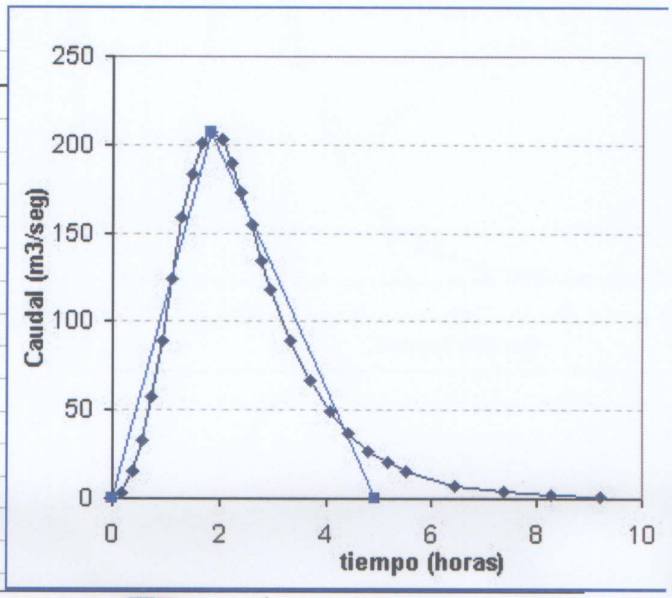
Volumen total por el área bajo el hidrograma (area triángulo = Base x altura / 2):

1.828.664 m3

Volumen total (area cuenca X lámina agua caída):

1.829.308 m3

t / tp	Q / Qp	t	Q
0	0	0,00	0,00
0,1	0,015	0,18	3,10
0,2	0,075	0,37	15,49
0,3	0,16	0,55	33,05
0,4	0,28	0,74	57,83
0,5	0,43	0,92	88,81
0,6	0,6	1,11	123,93
0,7	0,77	1,29	159,04
0,8	0,89	1,47	183,82
0,9	0,97	1,66	200,35
1	1	1,84	206,54
1,1	0,98	2,03	202,41
1,2	0,92	2,21	190,02
1,3	0,84	2,39	173,50
1,4	0,75	2,58	154,91



1,5	0,65	2,76	134,25
1,6	0,57	2,95	117,73
1,8	0,43	3,32	88,81
2	0,32	3,68	66,09
2,2	0,24	4,05	49,57
2,4	0,18	4,42	37,18
2,6	0,13	4,79	26,85
2,8	0,098	5,16	20,24
3	0,075	5,53	15,49
3,5	0,036	6,45	7,44
4	0,018	7,37	3,72
4,5	0,009	8,29	1,86
5	0,004	9,21	0,83

UNAUTHORIZED USE OR DUPLICATION IS PROHIBITED
PROHIBIDA LA REPRODUCCION SIN AUTORIZACION
DEL AUTOR

Datos de entrada

Long cauce=	10,2	km.
Cota max=	257	m
Cota min=	15	m
Superficie=	27,7	km2
Precipitación=	81,28	mm
Duración P neta=	1,68	horas

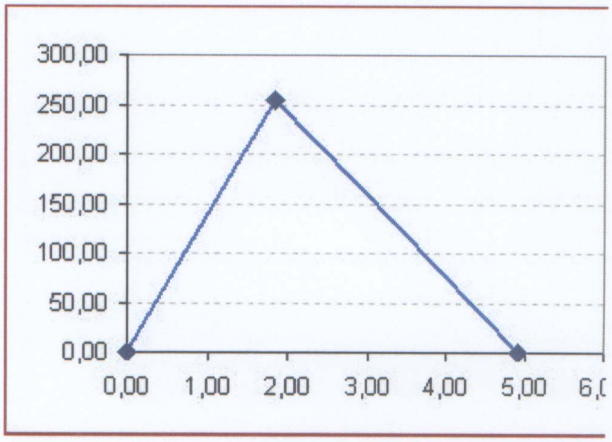
Cálculos

Pendiente=	0,0237255m/m
t conc=	100,2minutos
t conc=	1,67horas
tiempo punta=	**1,84**horas
tiempo base=	4,92horas
Caudal de la punta=	**254,21**m^3/seg.

Q50

Datos para dibujar el triángulo

tiempo	Q
0,00	0,00
1,84	254,21
4,92	0,00



Abajo puede verse la "suavización" del hidrograma triangular por el método del Hidrograma Adimensional del SCS

Comprobación:

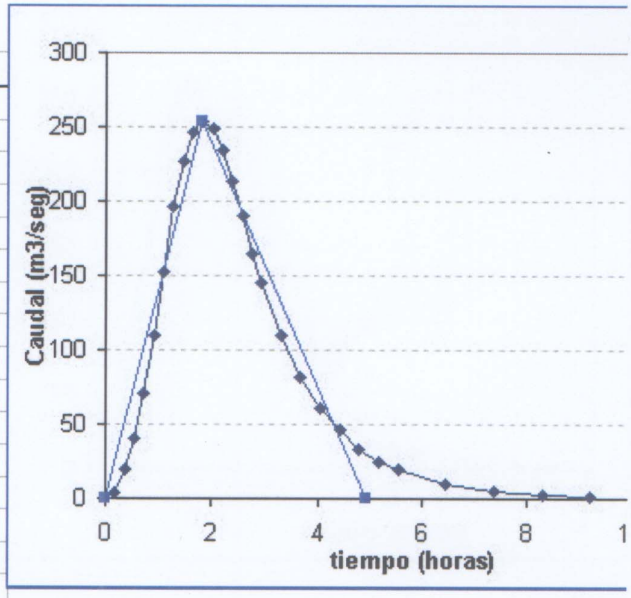
Volumen total por el área bajo el hidrograma (area triángulo = Base x altura / 2):

2.250.663 m3

Volumen total (area cuenca X lámina agua caída):

2.251.456 m3

t / tp	Q / Qp	t	Q
0	0	0,00	0,00
0,1	0,015	0,18	3,81
0,2	0,075	0,37	19,07
0,3	0,16	0,55	40,67
0,4	0,28	0,74	71,18
0,5	0,43	0,92	109,31
0,6	0,6	1,11	152,53
0,7	0,77	1,29	195,74
0,8	0,89	1,47	226,25
0,9	0,97	1,66	246,58
1	1	1,84	254,21
1,1	0,98	2,03	249,12
1,2	0,92	2,21	233,87
1,3	0,84	2,39	213,54
1,4	0,75	2,58	190,66
1,5	0,65	2,76	165,24
1,6	0,57	2,95	144,90
1,8	0,43	3,32	109,31
2	0,32	3,68	81,35
2,2	0,24	4,05	61,01
2,4	0,18	4,42	45,76



CENTRO DE RECURSOS TECNICOS
AUTORIDAD DEL CANAL DE PANAMA

2,6	0,13	4,79	33,05
2,8	0,098	5,16	24,91
3	0,075	5,53	19,07
3,5	0,036	6,45	9,15
4	0,018	7,37	4,58
4,5	0,009	8,29	2,29
5	0,004	9,21	1,02

Datos de entrada

Long cauce=	10,2	km.
Cota max=	257	m
Cota min=	15	m
Superficie=	27,7	km2
Precipitación=	83,82	mm
Duración P neta=	1,68	horas

Cálculos

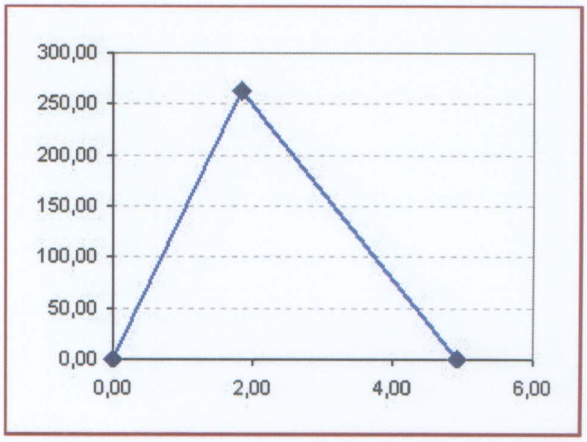
Pendiente=	0,02372549 m/m
t conc=	100,2 minutos
t conc=	1,67 horas
tiempo punta=	**1,84** horas
tiempo base=	4,92 horas
Caudal de la punta=	**262,15** m^3/seg.

Q100

Datos para dibujar el triángulo

tiempo	Q
0,00	0,00
1,84	262,15
4,92	0,00

Abajo puede verse la "suavización" del hidrograma triangular por el método del Hidrograma Adimensional del SCS



Comprobación:

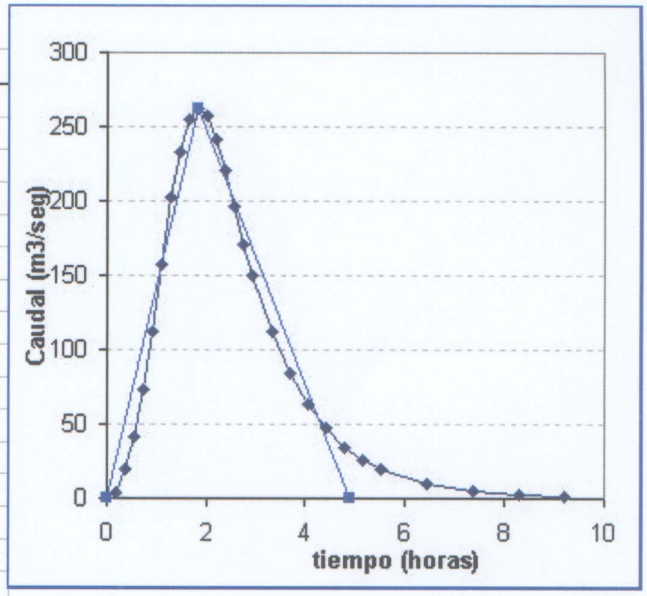
Volumen total por el área bajo el hidrograma (area triángulo = Base x altura / 2):

2.320.997 m3

Volumen total (area cuenca X lámina agua caída):

2.321.814 m3

t / tp	Q / Qp	t	Q
0	0	0,00	0,00
0,1	0,015	0,18	3,93
0,2	0,075	0,37	19,66
0,3	0,16	0,55	41,94
0,4	0,28	0,74	73,40
0,5	0,43	0,92	112,73
0,6	0,6	1,11	157,29
0,7	0,77	1,29	201,86
0,8	0,89	1,47	233,32
0,9	0,97	1,66	254,29
1	1	1,84	262,15
1,1	0,98	2,03	256,91
1,2	0,92	2,21	241,18
1,3	0,84	2,39	220,21
1,4	0,75	2,58	196,61
1,5	0,65	2,76	170,40
1,6	0,57	2,95	149,43
1,8	0,43	3,32	112,73
2	0,32	3,68	83,89
2,2	0,24	4,05	62,92
2,4	0,18	4,42	47,19
2,6	0,13	4,79	34,08



2,8	0,098	5,16	25,69
3	0,075	5,53	19,66
3,5	0,036	6,45	9,44
4	0,018	7,37	4,72
4,5	0,009	8,29	2,36
5	0,004	9,21	1,05

Datos de entrada

Long cauce=	10,2	km.
Cota max=	257	m
Cota min=	15	m
Superficie=	27,7	km2
Precipitación=	109,22	mm
Duración P neta=	1,68	horas

Cálculos

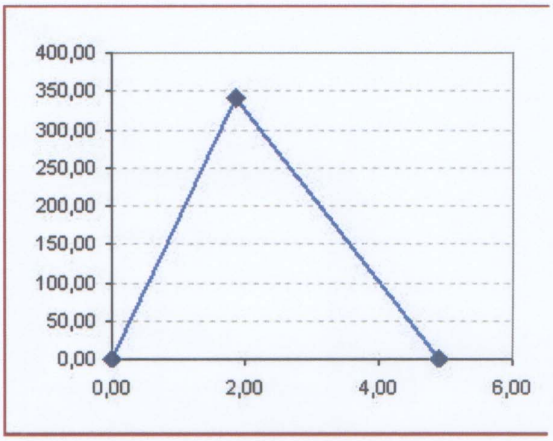
Pendiente=	0,02373 m/m
t conc=	100,2 minutos
t conc=	1,67 horas
tiempo punta=	**1,84** horas
tiempo base=	4,92 horas
Caudal de la punta=	**341,59** m^3/seg.

PMF

Datos para dibujar el triángulo

tiempo	Q
0,00	0,00
1,84	341,59
4,92	0,00

Abajo puede verse la "suavización" del hidrograma triangular por el método del Hidrograma Adimensional del SCS



Comprobación:

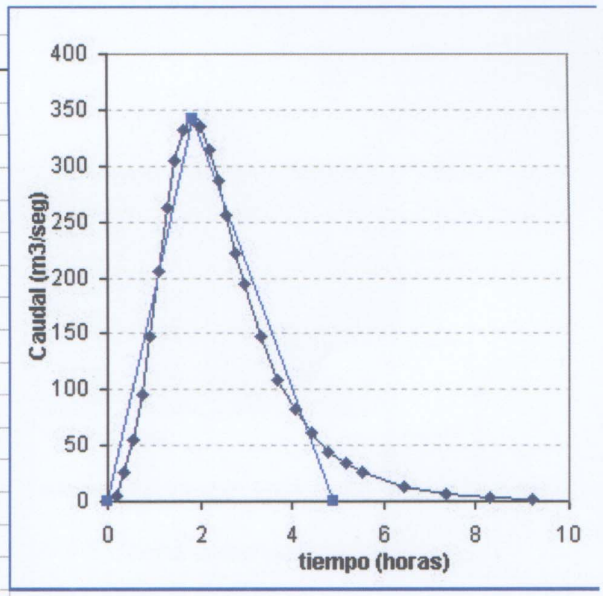
Volumen total por el área bajo el hidrograma (area triángulo = Base x altura / 2):

3.024.329 m3

Volumen total (area cuenca X lámina agua caída):

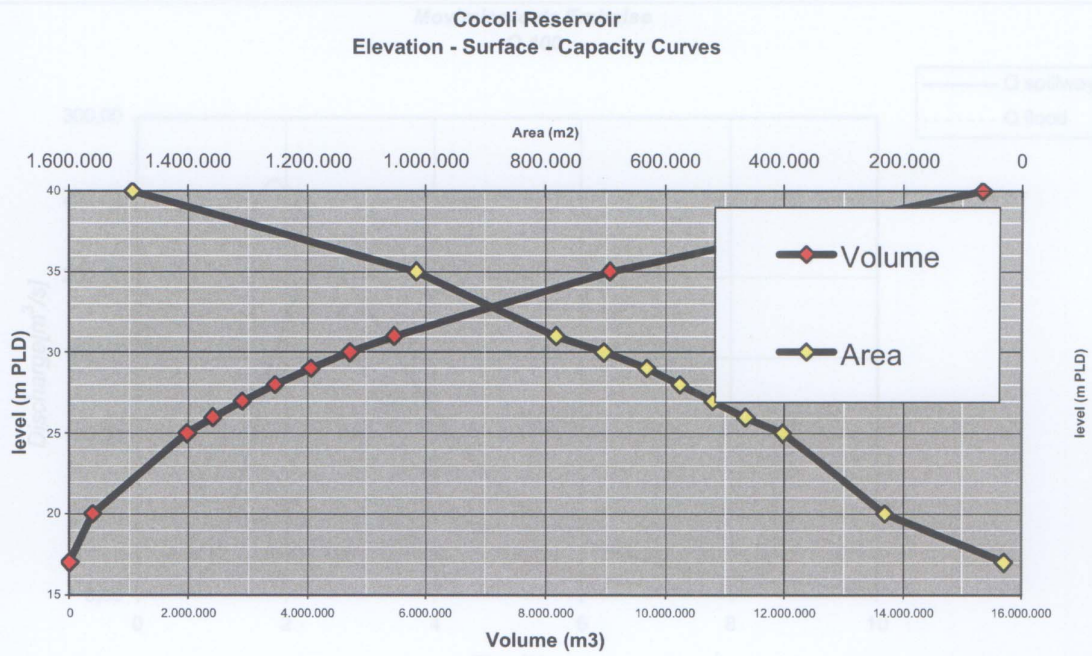
3.025.394 m3

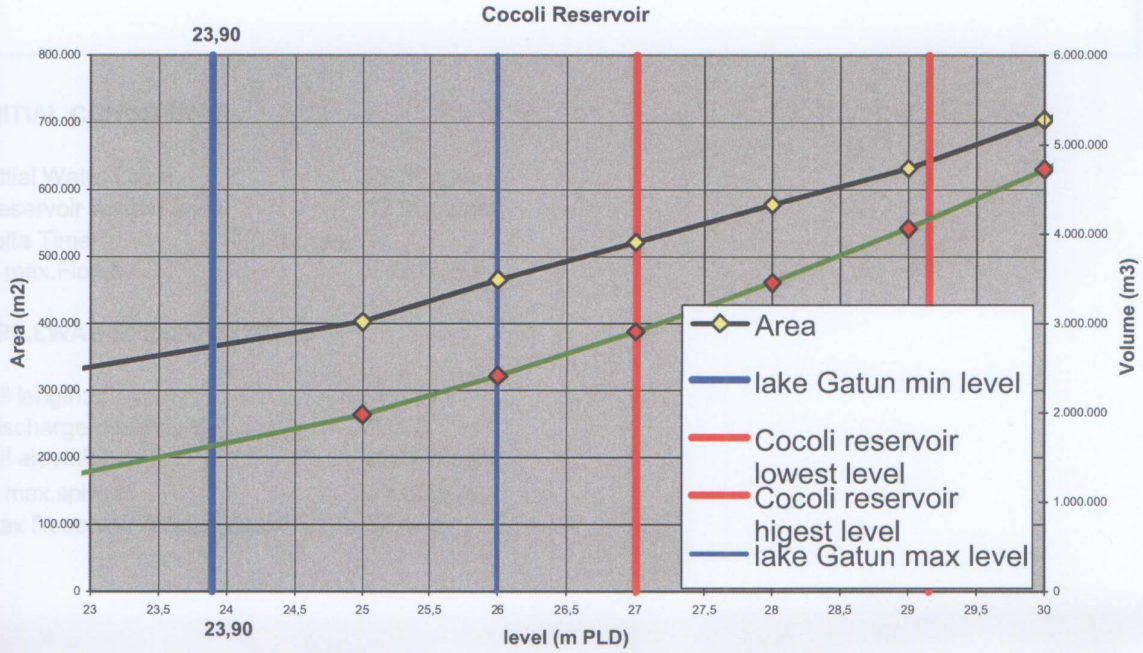
t / tp	Q / Qp	t	Q
0	0	0,00	0,00
0,1	0,015	0,18	5,12
0,2	0,075	0,37	25,62
0,3	0,16	0,55	54,65
0,4	0,28	0,74	95,65
0,5	0,43	0,92	146,88
0,6	0,6	1,11	204,96
0,7	0,77	1,29	263,03
0,8	0,89	1,47	304,02
0,9	0,97	1,66	331,34
1	1	1,84	341,59
1,1	0,98	2,03	334,76
1,2	0,92	2,21	314,27
1,3	0,84	2,39	286,94
1,4	0,75	2,58	256,19
1,5	0,65	2,76	222,04
1,6	0,57	2,95	194,71
1,8	0,43	3,32	146,88
2	0,32	3,68	109,31
2,2	0,24	4,05	81,98
2,4	0,18	4,42	61,49
2,6	0,13	4,79	44,41



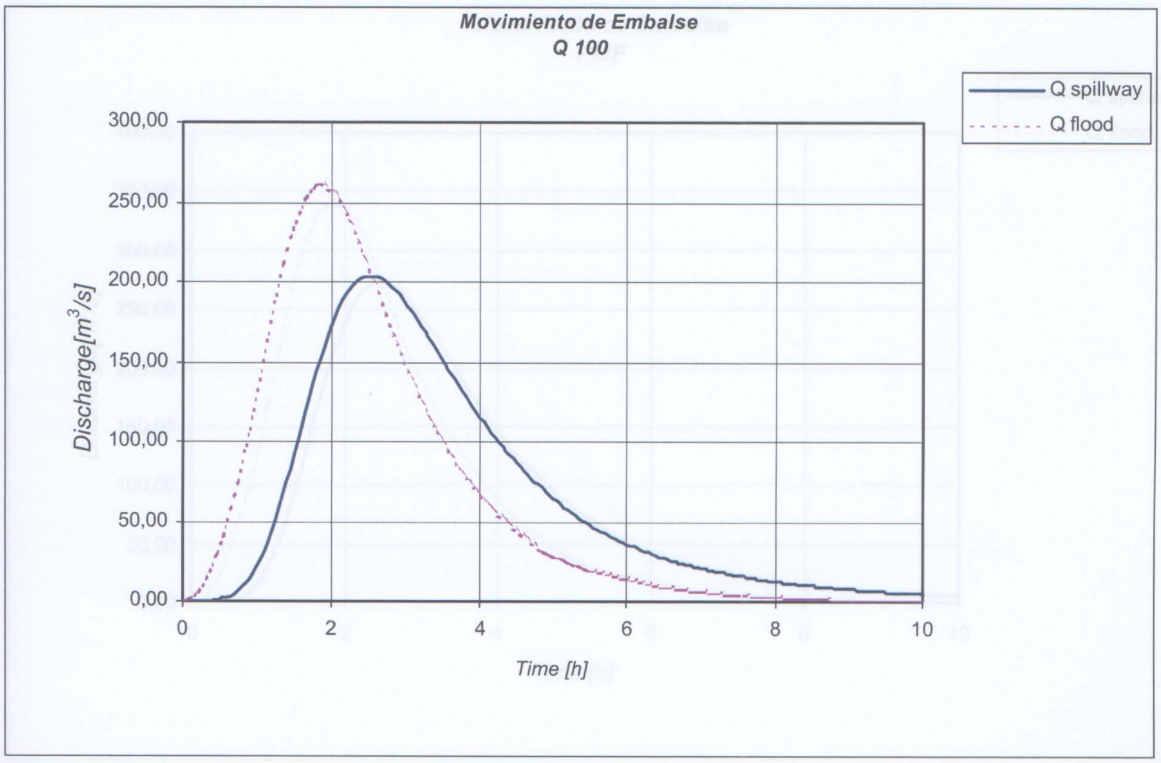
2,8	0,098	5,16	33,48
3	0,075	5,53	25,62
3,5	0,036	6,45	12,30
4	0,018	7,37	6,15
4,5	0,009	8,29	3,07
5	0,004	9,21	1,37

9.2 APPENDIX 2 : RIO COCOLI RESERVOIR – CAPACITY CURVES





9.3 APPENDIX 3 : RIO COCOLI RESERVOIR – FLOOD ROUTING Q100



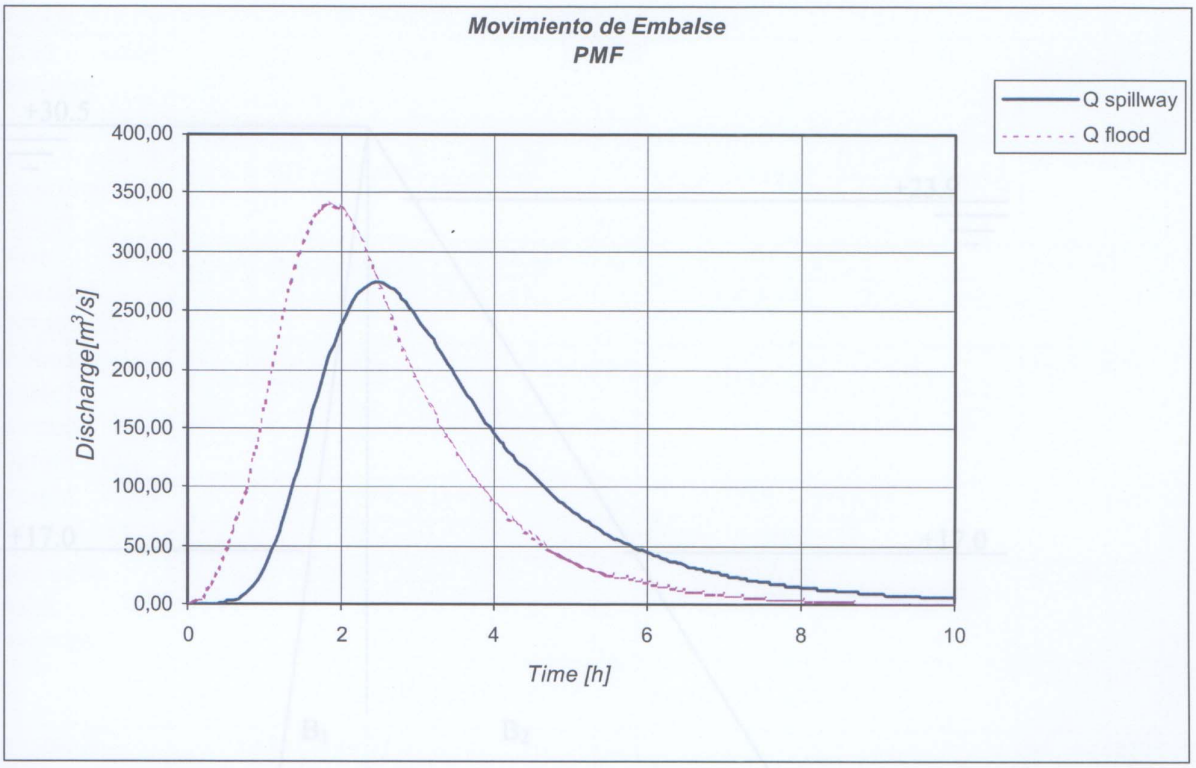
INITIAL CONDITIONS

Initial Water Level		29,50 msnm
Reservoir bottom level		17,00 msnm
delta Time =	0,05 horas	
Q max.Flood		262 m3/s

SPILLWAY SPECIFICATIONS

Sill length:	100 m
Discharge coefficient:	2,033
Sill elevation.:	29,50 msnm
Q max.spilled:	203,86 m^3/s
Max Reservoir Water level:	30,48 m

9.4 APPENDIX 4 : RIO COCOLI RESERVOIR – FLOOD ROUTING PMF



INITIAL CONDITIONS

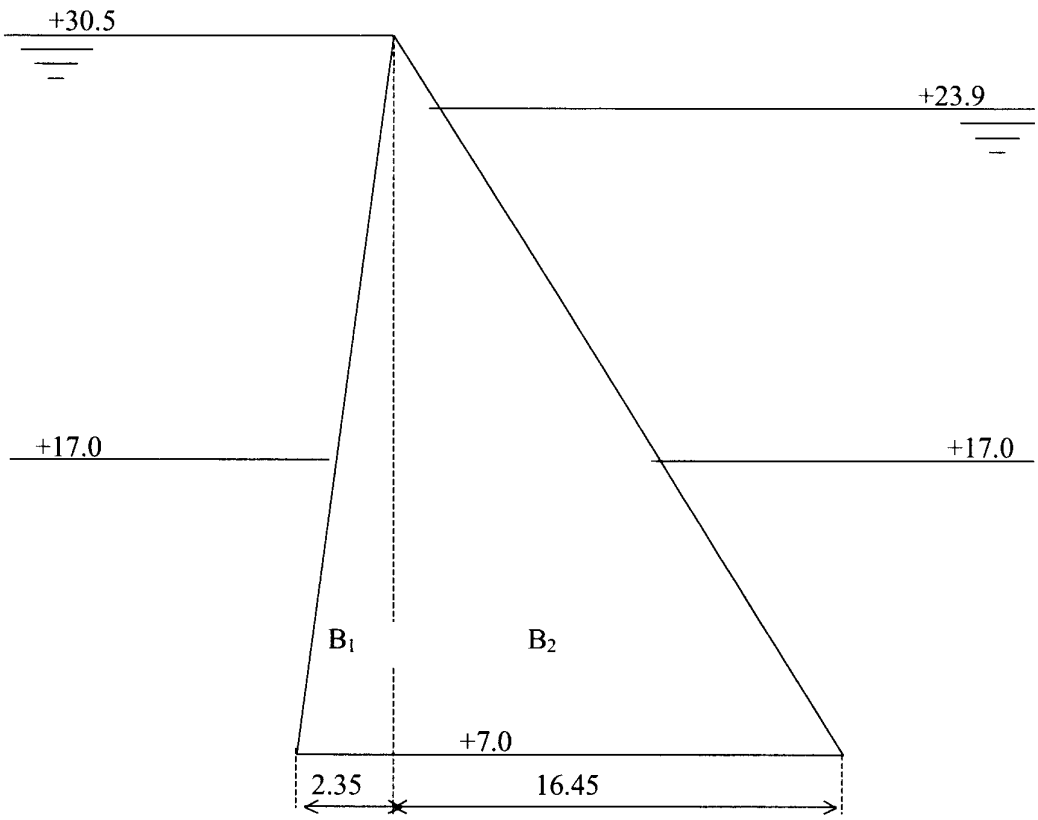
Initial Water Level		29,50 msnm
Reservoir bottom level		17,00 msnm
delta Time =	0,05 horas	
Q max.Flood		342 m3/s

SPILLWAY SPECIFICATIONS

Sill length:	100 m
Discharge coefficient:	2,033
Sill elevation.:	29,50 msnm
Q max.spilled:	273,73 m^3/s
Max Reservoir Water level:	30,68 m

9.5 APPENDIX 5 : RIO COCOLI DAM – GENERAL STABILITY

9.5.1 SCHEMATIC DRAWING

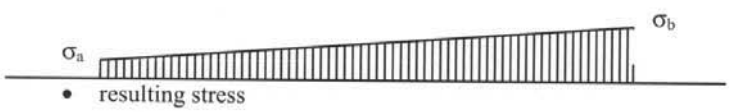


9.5.2 NORMAL CONDITIONS

identification	Calculation		Moment arm a	Moment
	[kN/m']		[m]	[kNm]
weight water Rio Cocoli	$W_0 = \frac{(30.5-7)*2.35}{2}*10$	↓	$a = 2.35*\frac{2}{3}+16.45$	M = –4975

identification	Calculation		Moment arm a	Moment
	$[kN/m']$		$[m]$	$[kNm]$
hydrostatic water pressure Rio Cocoli	$W_1 = \frac{(30.5-7)^2 * 10}{2}$	$\rightarrow$	$a = \frac{1}{3} * (30.5-7)$	$M = 21629$
Horz. earth pressure Rio Cocoli	$W_2 = \frac{(17-7)^2 * 10 * 0.5}{2}$	$\rightarrow$	$a = \frac{1}{3} * (17-7)$	$M = 833$
Horz. earth pressure Gatun Lake	$W_2' = -\frac{(17-7)^2 * 10 * 0.5}{2}$	$\leftarrow$	$a = \frac{1}{3} * (17-7)$	$M = -833$
Vert. earth pressure Rio Cocoli	$W_3 = \frac{1}{2} * (17-7) * 10 * \frac{(17-7) * 2.35}{(30.5-7)}$	$\downarrow$	$a = (2.35+16.4) - \frac{(17-7) * 2.35}{(30.5-7)} * \frac{1}{3}$	$M = -923$
Vert. earth pressure Gatun Lake	$W_3' = \frac{1}{2} * (17-7) * 10 * \frac{(17-7) * 16.45}{(30.5-7)}$	$\downarrow$	$a = \frac{(17-7) * 16.45}{(30.5-7)} * \frac{1}{3}$	$M = -817$
hydrostatic water pressure Gatum Lake	$W_4 = \frac{(23.9-7)^2 * 10}{2}$	$\leftarrow$	$a = \frac{1}{3} * (23.9-7)$	$M = -8044$
weight water Gatun Lake	$W_5 = \frac{1}{2} * \frac{(23.9-7)^2 * 16.45 * 10}{(30.5-7)}$	$\downarrow$	$a = \frac{1}{3} * \frac{(23.9-7) * 16.45}{(30.5-7)}$	$M = -3942$
hydraulic uplift pressure	$V_1 = (2.35+16.45) * (23.9-7) * 10$	$\uparrow$	$a = \frac{2.35+16.45}{2}$	$M = 29866$
	$V_2 = \frac{1}{2} * (2.35+16.45) * (30.5-23.9) * 10$	$\uparrow$	$a = \frac{2}{3} * (2.35+16.45)$	$M = 7775$
weight B_1	$B_1 = \frac{2.35 * (30.5-7)}{2} * 24$	$\downarrow$	$a = \frac{1}{3} * 2.35 + 16.45$	$M = -11420$
weight B_2	$B_2 = \frac{16.45 * (30.5-7)}{2} * 24$	$\downarrow$	$a = \frac{2}{3} * 16.45$	$M = -50875$

- vertical force balance : $\sum \updownarrow = 3180 kN \downarrow$
- Horizontal force balance : $\sum \leftrightarrow = 1333 kN \rightarrow$
- Moment balance $\sum M = -21726 kNm$



- resulting stress

$$\left\{ \begin{array}{l} \frac{\sigma_a + \sigma_b}{2} * 18.8 = 3180 kN \\ \frac{\sigma_a * 18.8^2}{2} + \frac{(\sigma_b - \sigma_a) * 18.8^2}{6} = 21726 kNm \end{array} \right\} \Rightarrow \begin{array}{l} \sigma_a = 31 \, kN/m^2 \\ \sigma_b = 308 \, kN/m^2 \end{array}$$

$308 \, kN/m^2 = 0.308 \, MPa < 15 \, MPa \, (concrete)$

- shear-control
  friction ratio base $\mu = 0.7$
  $1333kN < 3180 * 0.7 = 2226kN$

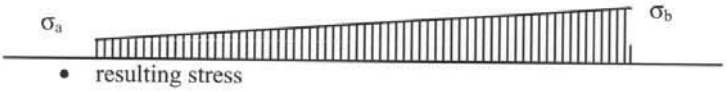
9.5.3 EARTHQUAKE CONDITIONS

- peak ground acceleration $a = 0.21 * g$
- effective seismic coefficient $k_h = 0.6 * \left(\frac{a_{max}}{g}\right) = 0.126$
- $k_v = 0$
- Dynamic water pressure during earthquake
  $P_{dw} \approx \frac{7}{12} * k_h * \gamma_w * H^2$
  - with
    - P_{dw} : dynamic water pressure force $\left(kN/m^2\right)$
    - k: seismic coëfficient
    - γ_w: unit weight of water
    - H: height of structure below still water level
    - $h_{dw} = \frac{3}{5} * H$ from water level

identification	Calculation		Moment arm a	Moment
	$\left[kN/m^2\right]$		[m]	[kNm]
Dynamic water pressure Rio Cocoli	$P_{dw} = \frac{7}{12} * 0.126 * 10 * (30.5 - 17)^2$	→	$a = \frac{2}{5} * (30.5 - 17) + (17 - 7)$	M = 7875
Dynamic water pressure Gatum Lake	$P_{dw} = \frac{7}{12} * 0.126 * 10 * (23.9 - 17)^2$	→	$a = \frac{2}{5} * (23.9 - 17) + (17 - 7)$	M = 447
weight water Rio Cocoli	$W_0 = \frac{(30.5 - 7) * 2.35}{2} * 10$	↓	$a = 2.35 * \frac{2}{3} + 16.45$	M = −4975
hydrostatic water pressure Rio Cocoli	$W_1 = \frac{(30.5 - 7)^2 * 10}{2}$	→	$a = \frac{1}{3} * (30.5 - 7)$	M = 21629
Horz. earth pressure Rio Cocoli	$W_2 = \frac{(17 - 7)^2 * 10 * 0.5}{2}$	→	$a = \frac{1}{3} * (17 - 7)$	M = 833
Horz. earth pressure	$W_2' = -\frac{(17 - 7)^2 * 10 * 1}{2}$	←	$a = \frac{1}{3} * (17 - 7)$	M = −1666

identification	Calculation		Moment arm a	Moment
	$[kN/m']$		$[m]$	$[kNm]$
Gatun Lake	$(\lambda_p = 1)$			
Vert. earth pressure Rio Cocoli	$W_3 = \frac{1}{2}*(17-7)*10*\frac{(17-7)*2.35}{(30.5-7)}$	$\downarrow$	$a = (2.35+16.4) - \frac{(17-7)*2.35}{(30.5-7)}*\frac{1}{3}$	$M = -923$
Vert. earth pressure Gatun Lake	$W_3' = \frac{1}{2}*(17-7)*10*\frac{(17-7)*16.45}{(30.5-7)}$	$\downarrow$	$a = \frac{(17-7)*16.45}{(30.5-7)}*\frac{1}{3}$	$M = -817$
horz.. earth pressure Rio Cocoli earthquake	$0.126*W_3$	$\rightarrow$	$a = \frac{2}{3}*(17-7)$	$M = 42$
horz.. earth pressure Gatun Lake earthquake	$0*W_3'$	$\leftarrow$	$a = \frac{2}{3}*(17-7)$	$M = -0$
hydrostatic water pressure Gatun Lake	$W_4 = \frac{(23.9-7)^2*10}{2}$	$\leftarrow$	$a = \frac{1}{3}*(23.9-7)$	$M = -8044$
weight water Gatun Lake	$W_5 = \frac{1}{2}*\frac{(23.9-7)^2*16.45*10}{(30.5-7)}$	$\downarrow$	$a = \frac{1}{3}*\frac{(23.9-7)*16.45}{(30.5-7)}$	$M = -3942$
hydraulic uplift pressure	$V_1 = (2.35+16.45)*(23.9-7)*10$	$\uparrow$	$a = \frac{2.35+16.45}{2}$	$M = 29866$
	$V_2 = \frac{1}{2}*(2.35+16.45)*(30.5-23.9)*10$	$\uparrow$	$a = \frac{2}{3}*(2.35+16.45)$	$M = 7775$
weight B_1	$B_1 = \frac{2.35*(30.5-7)}{2}*24$	$\downarrow$	$a = \frac{1}{3}*2.35+16.45$	$M = -11420$
weight B_2	$B_2 = \frac{16.45*(30.5-7)}{2}*24$	$\downarrow$	$a = \frac{2}{3}*16.45$	$M = -50875$
horz. comp. weight during earthquake	$0.126*B_1$	$\rightarrow$	$a = \frac{1}{3}*(30.5-7)$	$M = 654$
	$0.126*B_2$	$\rightarrow$	$a = \frac{1}{3}*(30.5-7)$	$M = 4578$

- vertical force balance : $\sum\updownarrow = 3180kN \downarrow$
- Horizontal force balance : $\sum\leftrightarrow = 2198kN \rightarrow$
- Moment balance $\sum M = -8963kNm$



- resulting stress

$$\left.\begin{cases}\frac{\sigma_a+\sigma_b}{2}*18.8=3180\,\text{kN}\\ \frac{\sigma_a*18.8^2}{2}+\frac{(\sigma_b-\sigma_a)*18.8^2}{6}=8963\,\text{kNm}\end{cases}\right\}\Rightarrow \begin{matrix}\sigma_a=-186\,\text{kN}/\text{m}^2\\ \sigma_b=524\,\text{kN}/\text{m}^2\end{matrix}$$

Length compression zone $x=\frac{18.8*524}{524+186}=13.87\,\text{m}$

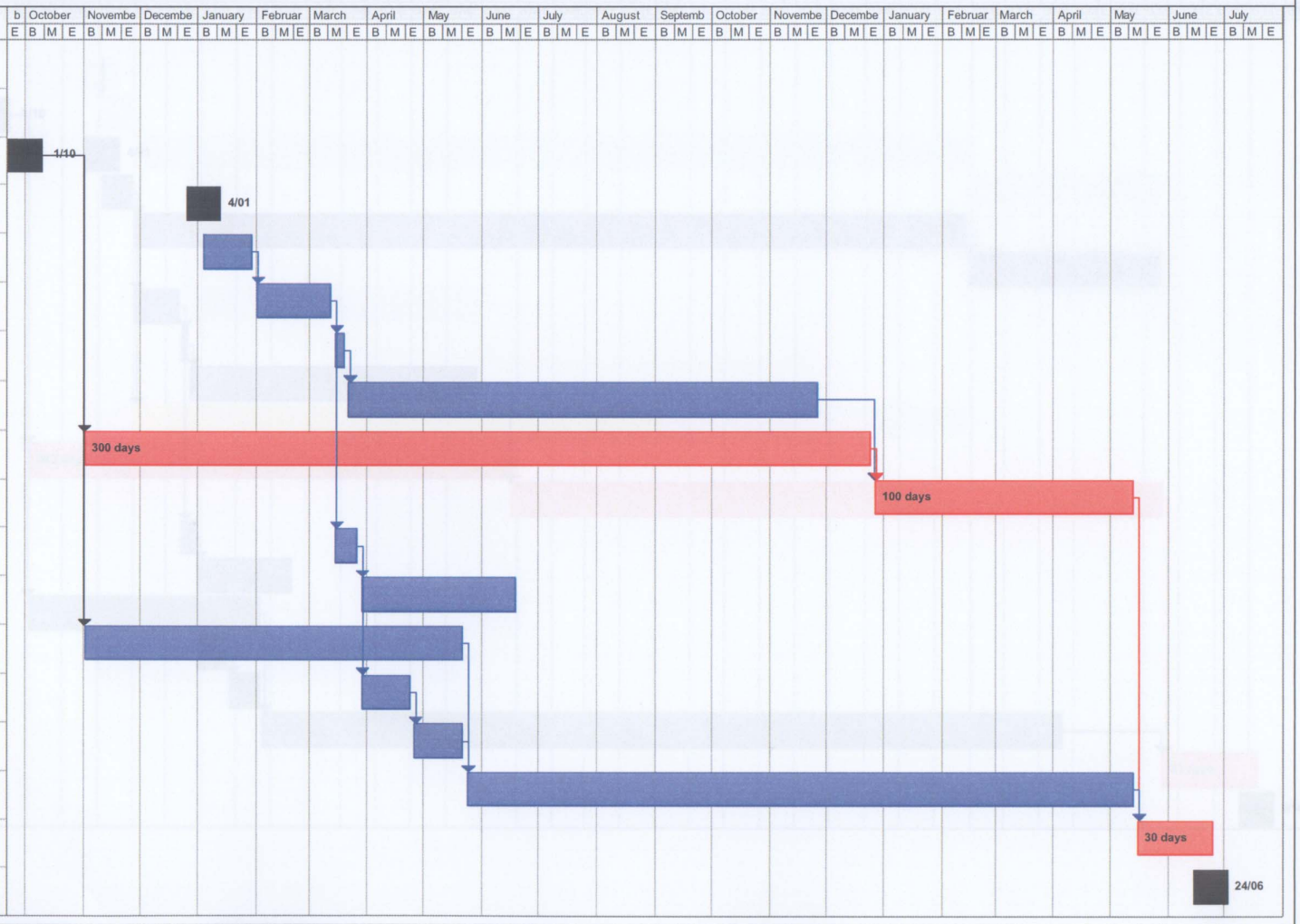
compression zone > 10% base $\Rightarrow$ $13.87\,\text{m}>0.1*18.8=1.88\,\text{m}$

- shear-control
  friction ratio base $\mu=0.7$
  $2198\,\text{kN}<3180*0.7=2226\,\text{kN}$

9.6 APPENDIX 6 : CONSTRUCTION SCHEDULE – GANTT CHARTS

APPENDIX 6.1 : DIRECT PUMPING
First Phase (3 pumps - one penstock)

ID	Task Name
1	
2	
3	Contract Award
4	Start Construction Work
5	Mobilization
6	Excavation for Pumping Station
7	Ground water lowering
8	Concrete works PS
9	Procurement Pumps + EM
10	Installation Pumps + EM
11	Excavation Discharge Structure
12	Concrete works DS
13	Procurement pipes for Penstock
14	Clearing for Penstock Startup
15	Excavation for Penstock Startup
16	Pipe erection + backfill
17	Testing EM
18	Completion of the work

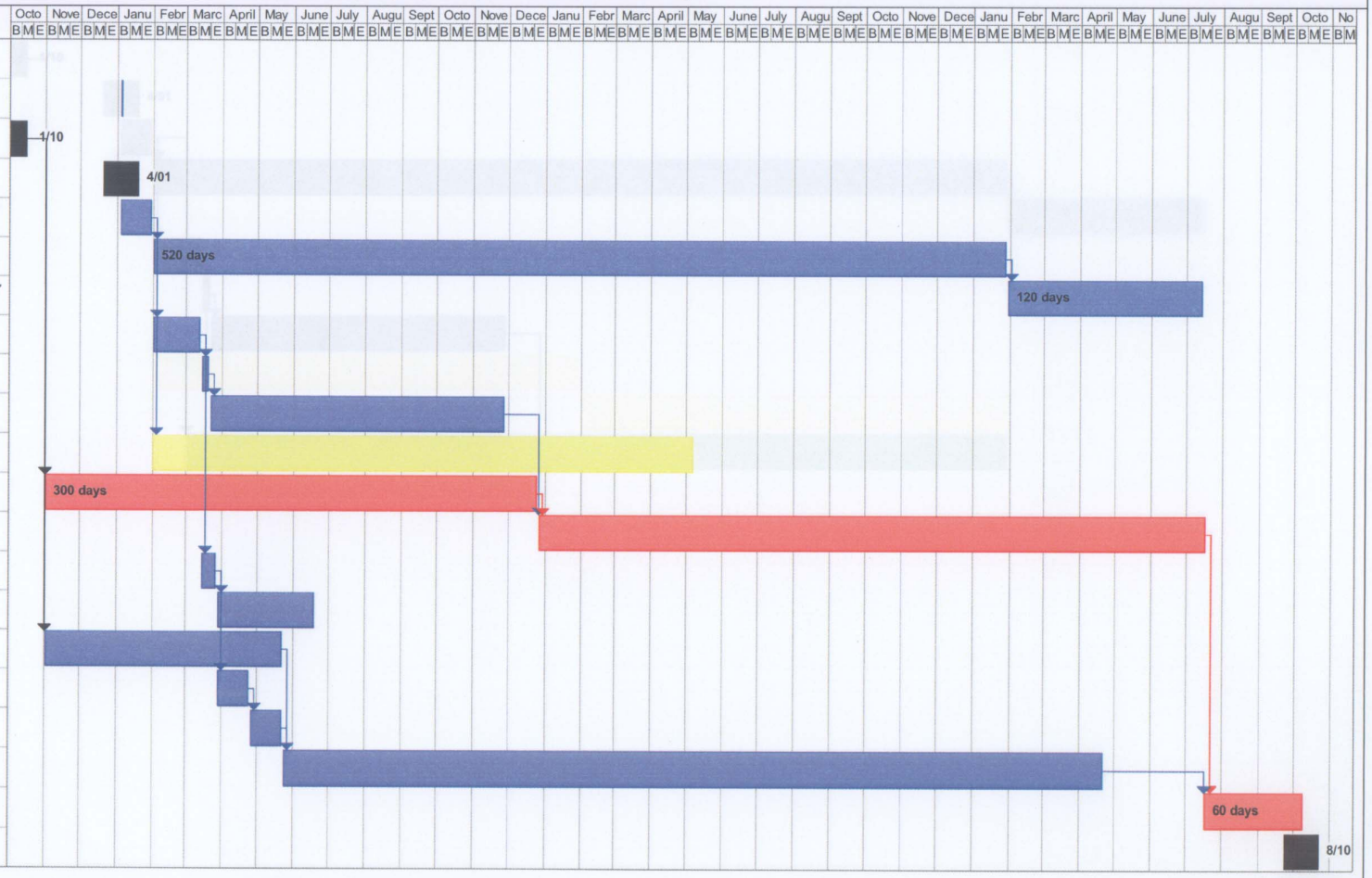


Project: DirectPumping
Date: Fri 21/11/03

Task	Summary	Rolled Up Progress	Project Summary
Progress	Rolled Up Task	Split	Group By Summary
Milestone	Rolled Up Milestone	External Tasks	

First Phase (3 pumps - one penstock)

ID	Task Name
1	
2	
3	Contract Award
4	Start Construction Work
5	Mobilization
6	Excavation Lower pond
7	Side slope protection and concrete layer
8	Excavation for Pumping Station
9	Ground water lowering
10	Concrete works PS
11	Conduit LC to Lower pond
12	Procurement Pumps + EM
13	Installation Pumps + EM
14	Excavation Discharge Structure
15	Concrete works DS
16	Procurement pipes for Penstock
17	Clearing for Penstock Startup
18	Excavation for Penstock Startup
19	Pipe erection + backfill
20	Testing EM - Filling Pond
21	Completion of the work

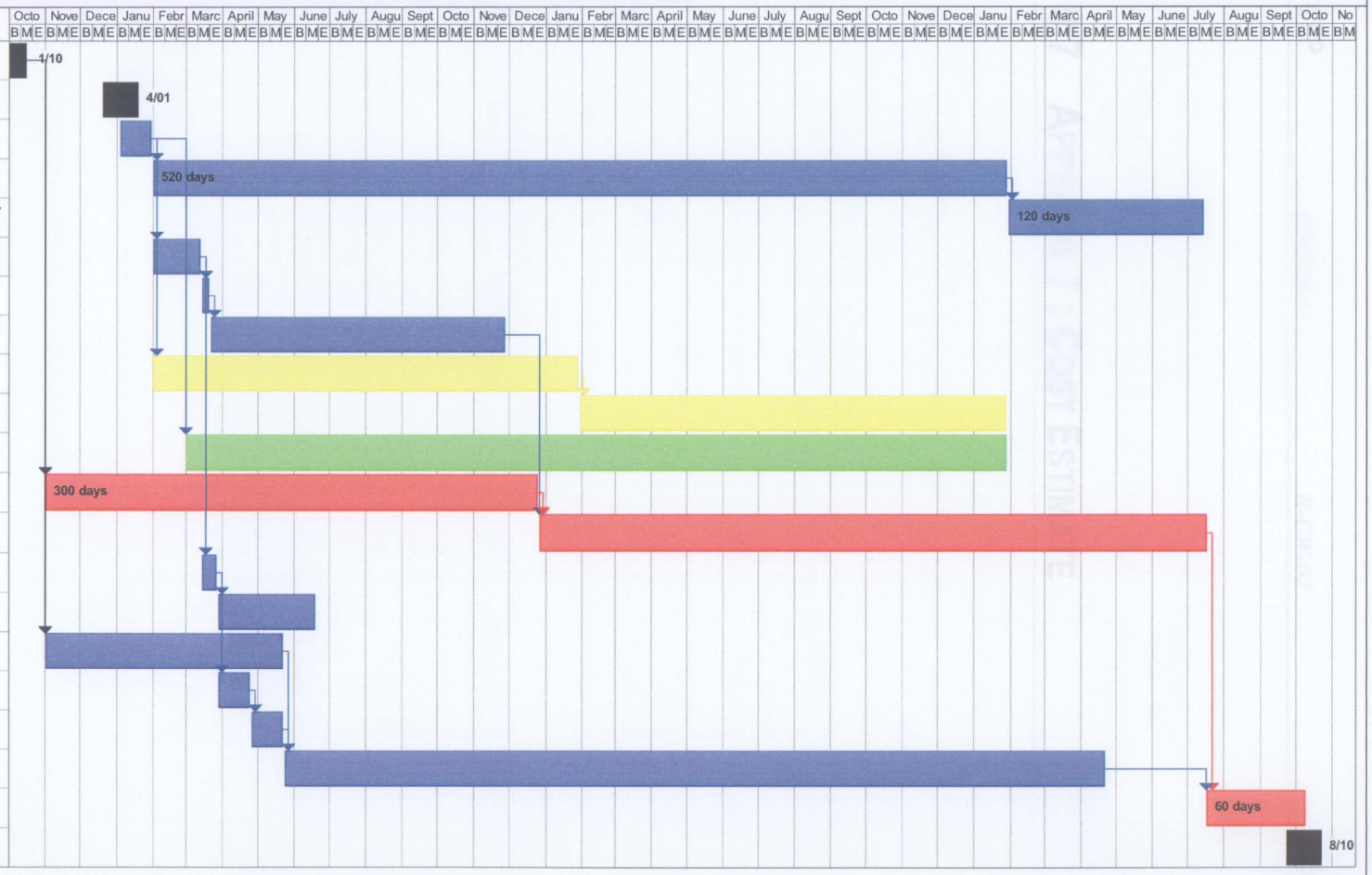


Project: SemiDirectPumping
Date: Wed 3/12/03

Task | Milestone | Rolled Up Task | Rolled Up Progress | External Tasks | Group By Summary
Progress | Summary | Rolled Up Milestone | Split | Project Summary

First Phase (3 pumps - one penstock)

ID	Task Name
1	Contract Award
2	Start Construction Work
3	Mobilization
4	Excavation Lower pond
5	Side slope protection and concrete layer
6	Excavation for Pumping Station
7	Ground water lowering
8	Concrete works PS
9	Conduit LC to Lower pond
10	Conduit LC to Upper pond
11	Rio Cocoli Reservoir + Dam
12	Procurement Pumps + EM
13	Installation Pumps + EM
14	Excavation Discharge Structure
15	Concrete works DS
16	Procurement pipes for Penstock
17	Clearing for Penstock Startup
18	Excavation for Penstock Startup
19	Pipe erection + backfill
20	Testing EM - Filling Pond
21	Completion of the work



Project: PondtopondPumping
Date: Fri 21/11/03

Task | Milestone | Rolled Up Task | Rolled Up Progress | External Tasks | Group By Summary
Progress | Summary | Rolled Up Milestone | Split | Project Summary

9.7 APPENDIX 7 : COST ESTIMATE

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Locks Emptying system for 3-step

Outlet construction in Lower Pond

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Concrete	m³	4.622	150	693.225
2	Reinforcement for outlet construction	kg	346.613	1,2	415.935
3	Excavation in overburden	m³	22.680	3	68.040
4	Backfill	m³	6.615	5	33.075
5	Bulkheads	kg	60.000	5	300.000
					1.510.275

Part to be constructed within Third Lane

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Extra Excavation	m³	381.075	6	2.286.450
2	Reinforced Concrete for culverts	m³	10.530	120	1.263.600
3	Reinforcement for culverts	kg	789.750	0,875	691.031
4	Reinforced Concrete for valve chamber	m³	24.470	150	3.670.500
5	Reinforcement for valve chamber	kg	1.835.250	0,875	1.605.844
6	Bulkheads	kg	460.000	5	2.300.000
7	Valves	kg	254.000	5	1.270.000
8	Operating machinery for valves	pcs	4	200.000	800.000
9	Backfill	m³	285.025	3	855.075
10	Ladders	pcs	8	2.000	16.000
					14.758.500

Culverts in open excavation (1034m)

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	543.372	3	1.630.116
2	Excavation in rock	m³	1.149.282	10	11.492.820
3	Reinforced concrete	m³	135.160	170	22.977.200
4	Reinforcement	kg	10.137.000	1,2	12.164.400
5	Backfill	m³	1.389.934	3	4.169.801
6	RCC	m³	167.426	40	6.697.020
					59.131.357

Culverts by boring (1034m)

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Bored tunnel	m	2.068	25.000	51.700.000
					51.700.000

Locks emptying system

Culverts in open excavation (1034m)	**75.400.132**
Culverts by boring (1034m)	**67.968.775**

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Locks Filling system for 3-step

Part to be constructed within Third Lane

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Extra Excavation	m³	386.125	6	2.316.750
2	Reinforced Concrete for culverts	m³	10.530	120	1.263.600
3	Reinforcement for culverts	kg	789.750	0,875	691.031
4	Reinforced Concrete for valve chamber	m³	24.381	150	3.657.150
5	Reinforcement for valve chamber	kg	1.828.575	0,875	1.600.003
6	Bulkheads	kg	460.000	5	2.300.000
7	Valves	kg	254.000	5	1.270.000
8	Operating machinery for valves	pcs	4	200.000	800.000
9	Backfill	m³	289.210	3	867.630
10	Ladders	pcs	8	2.000	16.000
					14.782.164

Culverts in open excavation (940m)

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	466.800	3	1.400.400
2	Excavation in rock	m³	527.850	10	5.278.500
3	Reinforced concrete	m³	106.857	170	18.165.690
4	Reinforcement	kg	8.014.275	1,2	9.617.130
5	Backfill	m³	779.043	3	2.337.129
6	RCC	m³	108.615	40	4.344.600
					41.143.449

Culverts by boring (940m)

Item	Description	Unit	Quantity	Unit cost	Total cost
	Bored tunnel	m	1.880	25.000	47.000.000
					47.000.000

Locks filling system

Culverts in open excavation (940m)	**55.925.613**
Culverts by boring (940m)	**61.782.164**

Locks Emptying system for 2-step

Outlet construction in Lower Pond

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Concrete	m³	4.622	150	693.225
2	Reinforcement for outlet construction	kg	346.613	1,2	415.935
3	Excavation in overburden	m³	22.680	3	68.040
4	Backfill	m³	6.615	5	33.075
5	Bulkheads	kg	60.000	5	300.000
					1.510.275

Part to be constructed within Third Lane

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Extra Excavation	m³	381.075	6	2.286.450
2	Reinforced Concrete for culverts	m³	10.530	120	1.263.600
3	Reinforcement for culverts	kg	789.750	0,875	691.031
4	Reinforced Concrete for valve chamber	m³	24.470	150	3.670.500
5	Reinforcement for valve chamber	kg	1.835.250	0,875	1.605.844
6	Bulkheads	kg	460.000	5	2.300.000
7	Valves	kg	254.000	5	1.270.000
8	Operating machinery for valves	pcs	4	200.000	800.000
9	Backfill	m³	285.025	3	855.075
10	Ladders	pcs	8	2.000	16.000
					14.758.500

Culverts in open excavation (1227m)

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	624.432	3	1.873.296
2	Excavation in rock	m³	1.242.694	10	12.426.940
3	Reinforced concrete	m³	155.473	170	26.430.453
4	Reinforcement	kg	11.660.494	1,2	13.992.593
5	Backfill	m³	1.517.458	3	4.552.375
6	RCC	m³	194.060	40	7.762.380
					67.038.036

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Culverts by boring (1227m)

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Bored tunnel	m	2.454	25.000	61.350.000
					61.350.000

Locks emptying system for 2-step

Culverts in open excavation (1227m)	**83.306.811**
Culverts by boring (1227m)	**77.618.775**

Tractebel Development Engineering | TECHNUM | CNR PROJETS | COYNE ET BELLIER

Locks Filling system for 2-step

Part to be constructed within Third Lane

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Extra Excavation	m³	386.125	6	2.316.750
2	Reinforced Concrete for culverts	m³	10.530	120	1.263.600
3	Reinforcement for culverts	kg	789.750	0,875	691.031
4	Reinforced Concrete for valve chamber	m³	24.381	150	3.657.150
5	Reinforcement for valve chamber	kg	1.828.575	0,875	1.600.003
6	Bulkheads	kg	460.000	5	2.300.000
7	Valves	kg	254.000	5	1.270.000
8	Operating machinery for valves	pcs	4	200.000	800.000
9	Backfill	m³	289.210	3	867.630
10	Ladders	pcs	8	2.000	16.000
					14.782.164

Culverts in open excavation (1125m)

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	536.175	3	1.608.525
2	Excavation in rock	m³	527.850	10	5.278.500
3	Reinforced concrete	m³	126.328	170	21.475.803
4	Reinforcement	kg	9.474.619	1,2	11.369.543
5	Backfill	m³	816.737	3	2.450.210
6	RCC	m³	120.825	40	4.833.000
					47.015.580

Culverts by boring (1125m)

Item	Description	Unit	Quantity	Unit cost	Total cost
	Bored tunnel	m	2.250	25.000	56.250.000
					56.250.000

Locks filling system for 2-step

Culverts in open excavation (1125m)	**61.797.745**
Culverts by boring (1125m)	**71.032.164**

Tractebel Development Engineering TECHNUM CNR PROJETS COYNE ET BELLIER

Lower Pond

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	3.608.891	3	10.826.673
2	Excavation in rock	m³	1.894.460	10	18.944.600
3	Concrete equalisation layer	m³	129.000	90	11.610.000
4	Concrete slab	m³	3.924	170	667.079
5	Anchored shotcrete	m²	15.049	100	1.504.941
6	Open stone asphalt mattress 15 cm	m²	25.991	115	2.989.012
7	Geotextile	m²	37.991	1,8	68.385
8	Drain	m	2.013	25	50.322
9	Drain material (gravel)	m³	662	6	3.975
10	Pavement	m²	19.978	24	479.482
11	Vegetated slope	m²	26352	0,5	13.176
					47.157.643

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Penstock Scenario 1 : Direct Pumping

outlet construction

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Concrete for outlet construction	m³	1.550	150	232.500
2	Reinforcement for outlet construction	kg	77.500	1,2	93.000
3	Excavation in rock for outlet	m³	12.367	10	123670
					449.170

2 lift without WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	51.882	3	155.646
2	Excavation in rock	m³	371.000	10	3.710.000
3	Sand backfill	m³	36.966	10	369.664
4	Sand cement	m³	5.788	40	231.526
5	Backfill	m³	9.127	5	45.637
6	Tubular steel pipe 3200/16mm	kg	3.055.468	5	15.277.338
7	Bulkheads	kg	5.000	5	25.000
8	Ladder	pcs	1	500	500
				for 1 pipe :	**19.815.311**

3 lift without WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	45.656	3	136.968
2	Sand backfill	m³	31.616	10	316.160
3	Sand cement	m³	5.350	40	214.016
4	Backfill	m³	14.916	5	74.578
5	Tubular steel pipe 2900/14mm	kg	2.423.293	5	12.116.467
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**12.883.689**

2 lift + WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	34.761	3	104.283
2	Sand backfill	m³	22.131	10	221.312
3	Sand cement	m³	4.621	40	184.832
4	Backfill	m³	25.130	5	125.650
5	Tubular steel pipe 2300/12mm	kg	1.646.732	5	8.233.658
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**8.895.235**

3 lift + WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	26.979	3	80.937
2	Sand backfill	m³	15.492	10	154.918
3	Sand cement	m³	3.988	40	159.539
4	Backfill	m³	32.402	5	162.008
5	Tubular steel pipe 1800/9mm	kg	966.769	5	4.833.843
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**5.416.746**

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Penstock Scenario 2 : Semi Direct Pumping

outlet construction

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Concrete for outlet construction	m³	938	150	140.700
2	Reinforcement for outlet construction	kg	46.900	1	56.280
					196.980

2 lift without WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	77.684	3	233.052
2	Sand backfill	m³	32.650	10	326.496
3	Sand cement	m³	5.112	40	204.490
4	Backfill	m³	14.120	5	70.601
5	Tubular steel pipe 3200/16mm	kg	2.698.661	5	13.493.306
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**14.353.445**

3 lift without WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	68.654	3	205.962
2	Sand backfill	m³	27.924	10	279.240
3	Sand cement	m³	4.726	40	189.024
4	Backfill	m³	19.232	5	96.162
5	Tubular steel pipe 2900/14mm	kg	2.140.310	5	10.701.551
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**11.497.439**

2 lift + WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	52.512	3	157.536
2	Sand backfill	m³	19.547	10	195.468
3	Sand cement	m³	4.081	40	163.248
4	Backfill	m³	28.254	5	141.270
5	Tubular steel pipe 2300/12mm	kg	1.454.432	5	7.272.161
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**7.955.183**

3 lift + WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	41.054	3	123.162
2	Sand backfill	m³	13.683	10	136.828
3	Sand cement	m³	3.523	40	140.909
4	Backfill	m³	34.677	5	173.383
5	Tubular steel pipe 1800/9mm	kg	853.873	5	4.269.365
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**4.869.146**

Penstock Scenario 3 : Pond to Pond Pumping

outlet construction

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Concrete for outlet construction	m³	1.924	150	288.600
2	Reinforcement for outlet construction	kg	96.200	1	115.440
3	Sand backfill	m³	1.640	10	16.400
4	Sand cement	m³	432	40	17.280
5	Backfill	m³	1.944	5	9.720
					447.440

2 lift without WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	84.712	3	254.136
2	Sand backfill	m³	37.529	10	375.288
3	Sand cement	m³	5.876	40	235.049
4	Backfill	m³	11.091	5	55.454
5	Tubular steel pipe 3200/16mm	kg	3.168.540	5	15.842.700
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**16.788.126**

3 lift without WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	74.706	3	224.118
2	Sand backfill	m³	30.901	10	309.010
3	Sand cement	m³	5.229	40	209.176
4	Backfill	m³	15.752	5	78.758
5	Tubular steel pipe 2900/14mm	kg	2.368.490	5	11.842.452
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**12.689.014**

2 lift + WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	56.889	3	170.667
2	Sand backfill	m³	21.631	10	216.307
3	Sand cement	m³	4.516	40	180.652
4	Backfill	m³	25.735	5	128.675
5	Tubular steel pipe 2300/12mm	kg	1.609.490	5	8.047.452
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**8.769.253**

3 lift + WSB

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Excavation in overburden	m³	44.255	3	132.765
2	Sand backfill	m³	15.141	10	151.415
3	Sand cement	m³	3.898	40	155.931
4	Backfill	m³	32.842	5	164.211
5	Tubular steel pipe 1800/9mm	kg	944.905	5	4.724.525
6	Bulkheads	kg	5.000	5	25.000
7	Ladder	pcs	1	500	500
				for 1 pipe :	**5.354.347**

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Pumping Station Lower Pond

n°	Description	Unit	Quantity	Unit cost USD	Total cost USD
0	Mobilization	pcs	1	125000	125.000,00
1	Additional rock excavations	m³	6325	10	63.250,00
2	Bottom slab concrete	m³	752	100	75.200,00
3	Rear wall concrete	m³	765	170	130.050,00
4	Rear wall formwork	m²	765	25	19.125,00
5	Slabs, walls, columns concrete	m³	3829	170	650.930,00
6	Slabs, walls, columns formwork	m²	7658	25	191.450,00
7	Steel reinforcement bars	kg	534600	1,2	641.520,00
8	Tubular bored piles L=8m	pcs	15	3500	52.500,00
9	Tubular bored piles L=12m	pcs	10	4950	49.500,00
10	Watertight sealing	m²	1000	50	50.000,00
11	Railings	m	34	125	4.250,00
12	Manhole covers	pcs	20	125	2.500,00
13	Ladders	pcs	9	500	4.500,00
	Subtotal				**2.059.775,00**

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Pumping Station Pacific Entrance

n°	Description	Unit	Quantity	Unit cost USD	Total cost USD
0	Mobilization	pcs	1	125000	125.000,00
1	Excavation entrance to Pacific	m³	33800	5	169.000,00
2	Additional rock excavations	m³	10500	10	105.000,00
3	Bottom slab concrete	m³	1870	100	187.000,00
4	Rear wall concrete	m³	625	170	106.250,00
5	Rear wall formwork	m²	625	25	15.625,00
6	Slabs, walls, columns concrete	m³	5012	170	852.040,00
7	Slabs, walls, columns formwork	m²	10024	25	250.600,00
8	Steel reinforcement bars	kg	750700	1,2	900.840,00
9	Backfill	m³	10500	3	31.500,00
10	Concrete slab access platform	m³	75	150	11.250,00
11	Watertight sealing	m²	758	50	37.900,00
12	Substation building	m²	190	750	142.500,00
13	Manhole covers	pcs	20	125	2.500,00
14	Ladders	pcs	9	500	4.500,00
	Subtotal				**2.941.505,00**

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Upper Pond Dam on Rio Cocoli

Item	Description	Unit	Quantity	Unit cost	Total cost
1	Clearing	m²	700.000	1	700.000
2	Excavation in overburden	m³	188.095	3	564.284
3	Excavation in rock	m³	5.000	10	50.000
4	Reinforced Concrete	m³	30.560	150	4.583.989
5	RCC	m³	20.814	40	832.576
6	RCC treated with bedding mortar	m³	6.327	150	949.044
7	Steel Reinforcement	kg	1.835.000	1	2.202.000
8	Levelling Concrete	m³	896	150	134.343
9	Grout curtain	m	7.125	135	961.875
10	Consolidation grouting	m	8.175	120	981.000
11	Backfill material	m³	45.153	5	225.764
12	Bulkheads	kg	152.000	5	760.000
13	Half pipe drain ϕ 300	m	185	15	2.775
14	Trash rack 18,00 x 11,00 m	m²	772	600	463.200
15	Railing 1,30 m	m	185	125	23.125
16	Ladders	pcs	8	500	4.000
					13.437.975

9.8 LIST OF DRAWINGS

CW- 001	General layout
CW- 002	Upper pond dam on Rio Cocoli : planview
CW- 003	Upper pond dam on Rio Cocoli : upstream view and sections
CW- 004	Emptying system : locks to lower pond
CW- 005	Filling system : upper pond to locks
CW- 006	Lower pond
CW- 007	Scenario 3: Pond to Pond Pumping
CW- 008	Scenario 2 : Semi Direct Pumping
CW- 009	Scenario 1 : Direct Pumping
CW- 010	Pumping Station Lower Pond
CW- 011	Pumping Station Pacific
CW- 012	Upper pond Situation